FUGRO WEST, INC.

GEOTECHNICAL REPORT
SYCAMORE CREEK ENHANCEMENT PROJECT
SANTA BARBARA, CALIFORNIA

Prepared for:
PENFIELD & SMITH

July 2010
Fugro Job No. 3037.047

DRAFT

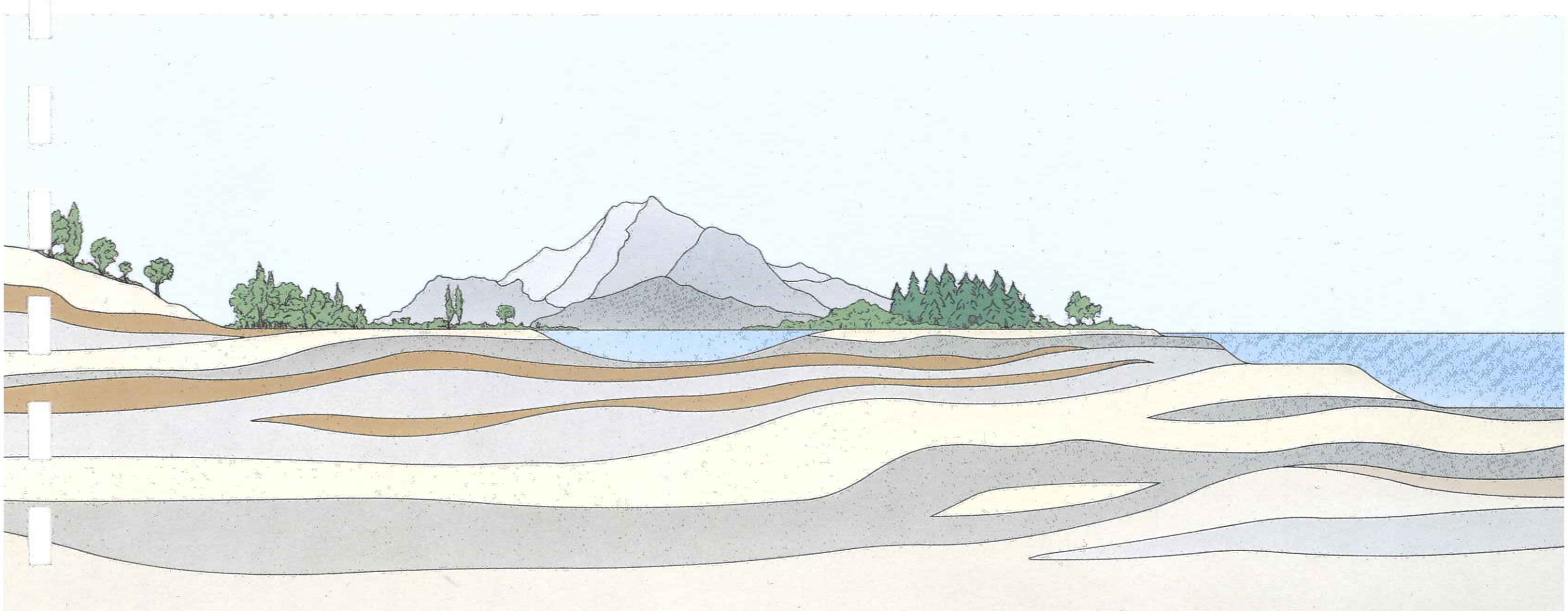


EXHIBIT N

FUGRO WEST, INC.

211 E. Victoria Street, Suite D
Santa Barbara, California 93101
Tel: (805) 963-4450
Fax: (805) 564-1327

July 25, 2010
Project No. 3037.047

Penfield & Smith Engineers
111 East Victoria Street
Santa Barbara, California 93101

Attention: Mr. Craig Steward

Subject: Geotechnical Report, Sycamore Creek Enhancement Project, Santa Barbara, California

Dear Mr. Steward:

Fugro is pleased to submit this geotechnical report for the Sycamore Creek Enhancement Project in Santa Barbara, Callifornia. This report was prepared according to our proposal dated November 30, 2009; and authorized by our Professional Services Agreement with Penfield & Smith on March 9, 2010.

We understand that improvements for this project will consist of widening and re-grading a section of the existing creek channel and replacing the bridge over Sycamore Creek at Punta Gorda Street. This report presents field and laboratory data collected during our evaluation and geotechnical opinions and recommendations for design of the proposed improvements.

We appreciate the opportunity to provide our services to Penfield & Smith on this project. Please call if we can provide further information or clarify any findings or recommendations.

Sincerely,

FUGRO WEST, INC.

Gregory S. Denlinger, G.E. 2249
Principal Engineer

Chad Stoehr, E.I.T. 114315
Staff Engineer II

DRAFT

Copies Submitted: (5 – Draft Copies) Addressee

CONTENTS

TABLES

PLATES

APPENDICES

1.0 INTRODUCTION

The planned project consists of widening and re-grading a section of the existing Sycamore Creek channel and replacing the existing bridge over Sycamore Creek at Punta Gorda Street in Santa Barbara, California. The location of the site relative to nearby streets and landmarks is shown on Plate 1 - Vicinity Map. The general layout of the site and proposed facilities are shown on Plate 2 - Field Exploration Plan. Our understanding of the project is based on discussions with you and an existing topographic base map of the site (Penfield & Smith, 2010).

1.1 SITE DESCRIPTION

The existing creek channel generally consists of a 10 to 15-foot-wide trapezoidal-shaped earthen channel with 5- to 8-foot-high side slopes inclined at about 1-1/2h:1v to about 3h:1v or flatter. However, some sections of the channel are vertical and retained by concrete or are sloped and have rip rap slope protection. The width of the channel at the top of the bank appears to range from about 30 feet to about 50 feet. At the bridge, Punta Gorda Street consists of a two-lane residential street with limited shoulder width. The existing bridge on Punta Gorda Street consists of an approximately 25- to 30-foot long single-span bridge with concrete wing walls and a concrete apron at the flow line of the creek. On the basis of information provided to us by Penfield & Smith, the existing bridge was constructed in about 1908 and is supported on gravity-type abutment walls with 3-foot-wide footings. The existing wingwalls appear to have been added to the bridge sometime after the initial construction of the bridge.

Highway 101 is located on the southeast end of the site and the surrounding land use consists mainly of single family residential housing. A mobile home park is present on the north side of the creek south of Punta Gorda Street. Sycamore Creek flows south underneath Highway 101 towards the Pacific Ocean. The existing ground surface along Sycamore Creek ranges from approximately elevation (el.) 25 feet near the crossing with Indio Muerto Street on the northwest end of the site, to approximately el. 15 feet near the crossing with Highway 101 on the southeast end of the site.

1.2 PROJECT DESCRIPTION

We understand that approximately 500 feet of the existing Sycamore Creek channel will be improved for the project. Conceptual plans for the project suggest the channel will be widened to the 60-foot-wide right of way limits with natural soil side slopes inclined at 2h:1v and rip rap slope protection placed at the toe of the channel slope. The current concept for the bridge replacement at Punta Gorda Street consists of removing the existing bridge and constructing a new bridge. The current approach to the project involves initial work consisting of improvements extending from the Caltrans right of way at Highway 101 and to a location about 120 feet downstream of Punta Gorda Street. A subsequent work effort will consist of channel improvements extending from the upstream limits of the initial work area to the concrete lined channel at Liberty Street and will include the bridge replacement at Punta Gorda Street.

2.0 WORK PERFORMED

2.1 PURPOSE

The purpose of this study was to characterize the geotechnical conditions along Sycamore Creek within the project limits including the bridge at Punta Gorda Street, and provide geotechnical opinions and recommendations for design of the channel improvements and bridge replacement.

2.2 SCOPE OF WORK

As a basis for providing the geotechnical recommendations presented in this report, we have performed the following scope of work:

- Site visits to observe the Punta Gorda Street bridge and conditions along the creek channel, mark the location of our explorations, and notifying underground service alert of the field exploration program;
- Field exploration program consisting of drilling one mud rotary wash boring, two hollow stem auger borings, and advancing four cone penetration test (CPT) soundings;
- Laboratory testing of selected samples obtained from the field exploration;
- Preparing a log of test borings sheet for the Punta Gorda Street bridge replacement; and
- Preparing this report summarizing the data obtained for the site, and our opinions and recommendations regarding;
    - Soil and groundwater conditions encountered, an assessment of geotechnical engineering parameters, and a design soil profile;
    - Grain size data for samples from the creek channel flow line and side slopes;
    - Geohazard assessment consisting of strong ground shaking and liquefaction potential;
    - Design of pile or cast-in-drill hole pile foundations, including suggested pile tip elevations, ultimate axial values and uplift capacities, and an evaluation of lateral pile loading using the computer program LPILE Plus.
    - Pile construction recommendations;
    - Site preparation and grading and general input to dewatering and temporary excavations and shoring (note: does not include design of groundwater dewatering or shoring system); and

- Site grading and general input to dewatering, temporary excavations, and temporary shoring (note: does not include design of groundwater dewatering or shoring system);
- Slope stability evaluations for the proposed creek bank geometry;
- Asphalt pavement structural section recommendations for reconstruction of Punta Gorda Street, and
- Corrosion potential of on-site soils.

We note that specific evaluations of liquefaction-induced lateral spreading, liquefaction impacts to slope stability, and scour potential are not a part of the geotechnical scope for this project.

2.3 FIELD EXPLORATION

The field exploration for this project consisted of advancing two (2) hollow stem auger borings, one (1) mud rotary wash boring, and four (4) cone penetration test (CPT) soundings. The logs of the borings are presented in Appendix A. The corresponding laboratory data is presented in Appendix B. The logs for the CPT soundings are presented in Appendix C. The approximate locations of the explorations are shown on Plate 2 – Field Exploration Plan.

2.3.1 Hollow Stem Auger Drilling

The drilling subcontractor for the project was S/G Drilling Company of Lompoc, California. S/G drilled two (2) hollow stem auger borings (DH-2 and DH-3) on April 22, 2010. The borings were drilled to depths of approximately 25 and 26 feet below the existing ground surface using 8-inch diameter hollow stem augers. The borings were sampled at approximate 5-foot intervals using a 3-inch outside diameter modified California split-spoon sampler or a Standard Penetration Test sampler. The modified California sampler was equipped with 1-inch high brass rings. The split-barrel samplers were driven into the materials at the bottom of the drill hole using a 140-pound automatic trip hammer with a 30-inch drop.

The blow count (N-value) shown on the boring logs is the number of blows from the hammer that were needed to drive the Standard Penetration Test (SPT) sampler one foot, after the sampler had been seated at least 6 inches into the material at the bottom of the hole[1]. Bulk samples were collected from the drill cuttings retrieved from the auger flights. Boring DH-2 performed outside of pavement was backfilled with the soil cuttings. Boring DH-3 performed in pavement was backfilled with the soil cuttings and capped with concrete dyed black. The sample intervals, N-values, a description of the subsurface conditions encountered and other field and laboratory data are presented on the logs of the borings in Appendix A.

[1] Sampler blow counts for the modified California sampler were also recorded during sampling. Equivalent SPT N-values for the modified California sampler were estimated by dividing the blow count by 1.6.

2.3.2 Mud Rotary Drilling

S/G Drilling excavated one mud rotary boring (DH-1) on April 23, 2010, near the existing Punta Gorda Street bridge. Boring DH-1 was advanced using rotary wash drilling techniques to drill a 6-inch diameter hole to a depth of approximately 101 feet below the existing ground surface. The upper 15 feet of the drill hole was excavated using hollow stem auger drilling methods and the auger was left in the hole to serve as casing for the rotary wash drilling.

The drill hole was generally sampled at approximately 5-foot intervals with the exception of two 10-foot intervals between depths of approximately 15 and 25 feet and 45 and 55 feet below the existing ground surface. The automatic hammer used was the same as that used for the hollow stem auger drilling. A 2-inch outside diameter standard penetration test (SPT) split-spoon sampler without liners was used in addition to the 3-inch outside diameter modified California split-spoon sampler used in the hollow stem auger borings. The log for the rotary wash drill hole is also provided in Appendix A.

2.3.3 Cone Penetration Testing

Fugro Geosciences of Santa Fe Springs, California advanced three (3) CPT soundings on March 31 and April 1, 2010 to depths ranging between approximately 49 and 82 feet below the existing ground surface. The CPT soundings were performed using an electric cone penetrometer. The penetrometer was advanced into the ground using a hydraulic ram mounted in a truck having a weight of approximately 20 tons. The cone penetrometer has a diameter of approximately 2 inches. Cone tip resistance (q_c), sleeve friction (f_s), and pore water pressures measured behind the tip (u_2) were recorded on the penetrometer during penetration. Data was recorded at approximately 2-centimeter intervals using an on-board computer to provide a near-continuous profile of the soil conditions encountered. The friction ratio (FR) was computed for each value of q_c and f_s recorded. The data was retrieved electronically for use in subsequent geotechnical analyses. CPT data and soil behavior type classifications were used to evaluate the subsurface conditions encountered at the site. The logs for the CPT soundings are presented in Appendix A.

2.4 LABORATORY TESTING

Laboratory testing for unit weight, moisture content, grain size distribution, fines content, direct shear, triaxial compression, and corrosion characteristics were performed as part of this program. Corrosion testing was performed by Cooper Testing Laboratory of Palo Alto, California. Laboratory testing was performed in general accordance with the applicable standards of ASTM. Laboratory test results are presented in Appendix B.

2.5 PREVIOUS STUDIES

We reviewed Log of Test Borings (LOTB's) for the Highway 101 overcrossing of Sycamore Creek and the soundwall located on the north side of Highway 101 (Caltrans 2007) near the project site. The subsurface information included on the LOTB's was used in addition to our own explorations in preparation of this report. The referenced LOTB's are provided in Appendix C – Caltrans 2007 LOTB's.

2.6 GENERAL CONDITIONS

Fugro prepared the conclusions and professional opinions presented in this report in accordance with generally accepted geotechnical principals and practices at the time and location this report was prepared. This statement is in lieu of all warranties, expressed or implied.

This report has been prepared for Penfield & Smith and their authorized agents only. It may not contain sufficient information for the purposes of other parties or other uses. If any changes are made in the project as described in this report, the conclusions and recommendations contained in this report should not be considered valid unless Fugro reviews the changes and modifies and approves, in writing, the conclusions and recommendations of this report. The report and drawings contained in this report are intended for design-input purposes; they are not intended to act as construction drawings or specifications.

Soil and rock deposits will vary in type, strength, and other geotechnical properties between points of observation and exploration. Additionally, groundwater and soil moisture conditions also can vary seasonally or for other reasons. Therefore, we do not and cannot have complete knowledge of the subsurface conditions underlying the site. The conclusions and recommendations presented in this report are based upon the findings at the points of exploration, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed during construction.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements or absence of statements, in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessment.

3.0 SITE CONDITIONS

3.1 GEOLOGIC SETTING

The project is situated in the Transverse Ranges geomorphic province of southern California. The Transverse Ranges province is oriented generally east-west, which is oblique to the general north-northwest trending structural trend of California mountain ranges. The Transverse Ranges province extends from the Los Angeles Basin westward to Point Arguello, and is composed of Cenozoic- to Mesozoic-age sedimentary, volcanic, igneous, and metamorphic rocks. The Santa Ynez Mountains and adjacent lowlands are comprised of sedimentary rocks and soil materials ranging in age from Cretaceous to recent.

Structural geology in the Santa Barbara and Goleta area consists of mountain and foothill areas underlain by generally south-dipping bedrock units and low lying coastal plain areas generally underlain by younger and older alluvium. The area generally includes a series of subparallel, east-west trending faults and folds that are the result of north-south compressional tectonics. The faults and folds roughly parallel the Santa Ynez Mountains and are present inland and offshore in the Santa Barbara Channel.

The general geology in the project area consists of a low-lying coastal plain of Quaternary-age alluvium unconformably overlying a thick sequence of Tertiary-age sedimentary rocks. Local geologic conditions in the project area as mapped by Dibblee (1986) are shown on Plate 3 – Regional Geologic Map.

3.2 SUBSURFACE CONDITIONS

The subsurface conditions at the site generally consist of shallow artificial fill overlying alluvial deposits (Qal). Locations of the explorations are provided on Plate 2 and the logs of the drill holes and CPT soundings advanced for this study are presented in Appendix A. A Log of Test Boring (LOTB) sheet prepared for the Punta Gorda Street bridge replacement is provided in Appendix D – Punta Gorda Street Bridge LOTB. A discussion of the soil conditions encountered within the artificial fill and alluvium is provided below.

3.2.1 Artificial Fill (Af)

A variable thickness of artificial fill was encountered in our explorations. We generally based our characterization of fill materials primarily on visual observation of the soil, the presence or absence of foreign material in the soil, consistency of the soil materials, and blow count data. However, it was difficult to distinguish artificial fill materials from the alluvial soils at this site because of similarities in color and soil type. Therefore, there is some uncertainty associated with the vertical extent of artificial fill logged in our explorations.

Artificial fill is logged in our drill holes extending from the ground surface to a depth of about 3 to 5 feet below the existing ground surface. The artificial fill material is likely associated with construction of the existing bridge at Punta Gorda Street and previous grading along Sycamore Creek. The artificial fill materials generally consisted of loose to medium dense silty and clayey sand and stiff sandy lean clay. Approximately 4 inches of asphalt concrete and 7 inches of base material were encountered in boring DH-2 performed in Soledad Street.

3.2.2 Alluvium (Qal)

Alluvial soils were encountered underlying the artificial fill materials in explorations performed at the site to the maximum depth explored, approximately 101 feet below the existing ground surface (DH-1).

Alluvial soils encountered in explorations DH-1 and CPT-3 performed on the south side of the bridge at Punta Gorda Street consist mainly of interbedded medium stiff to very stiff clay and silt and loose to medium dense sand with varying amounts of gravel to a depth of approximately 28 feet below the existing ground surface. A medium dense to dense sand stratum with varying gravel content was encountered between approximately 28 and 42 feet below the existing ground surface. That sand stratum is underlain by interbedded or stratified dense to very dense silty sand and stiff to very stiff clay.

Alluvial soils encountered in CPT-1 on the north side of the bridge consist of stratified medium dense to silty sand and medium stiff to very stiff clay and plastic silt to a depth of about 35 feet below the existing ground surface. A stratum of dense to very dense silty sand was encountered between a depth of 35 and 55 feet below the ground surface. That dense to very dense silty sand stratum is underlain by very stiff to hard clay encountered between 55 and 77 feet below the ground surface. CPT-1 met refusal in a dense to very dense sand stratum at a depth of 82 feet below the existing ground surface.

Alluvial soils encountered in explorations performed along Sycamore Creek (DH-2, DH-3, CPT-2, and CPT-4) consist mainly of medium stiff to very stiff clay and silt with varying sand content and occasional layers of medium dense sand extending to a depth of about 20 feet below the existing ground surface. Interbedded or stratified medium dense to very dense sand and firm to very stiff clay and silt was encountered below approximately 20 feet to the completion depths in CPT-2 and CPT-3 of approximately 50 feet below the existing ground surface.

3.3 GROUNDWATER CONDITIONS

A summary of the groundwater conditions encountered in our field exploration is provided below:

Table 1. Estimated Groundwater Measurements

Exploration Number	Approximate Depth to Groundwater (ft)	Approximate Groundwater Elevation (NAVD88)[1]	Notes
DH-1	8.5	+9.5	Depth to groundwater measured in hollow-stem auger before beginning mud rotary drilling
DH-2	7.5	+8.5	Depth to groundwater measured approximately 24 hours after completion of drilling.
DH-3	5.3	+15.7	Depth to groundwater measured in adjacent CPT-2
CPT-1	7.3	+12.7	Depth to groundwater measured after CPT rods were withdrawn
CPT-2	5.3	+14.7	Depth to groundwater measured after CPT rods were withdrawn
CPT-3	7	+10.5	Depth to groundwater measured after CPT rods

			were withdrawn
CPT-4	7.3	+8.7	Depth to groundwater measured after CPT rods were withdrawn
1) Based on ground surface elevations provided on Penfield & Smith topographic base map shown on Plate 2			

For the purposes of developing design recommendations for the bridge project, estimating liquefaction potential, and performing slope stability analysis for design of the channel improvements, the depth to groundwater in the project area was assumed to be 5 feet below the existing ground surface. We note that groundwater levels and zones of perched water in Sycamore Creek can vary over time in response to environmental changes and land use changes. In addition, groundwater levels in the area will likely fluctuate with the ocean tides. As such, groundwater levels at the time of construction or in the future could differ from the values reported in this study.

4.0 GEOLOGIC PROFILES AND SOIL PARAMETERS - PUNTA GORDA STREET

The subsurface profile used in our geotechnical analyses considered the soil conditions encountered during our field exploration on both sides of the bridge at Punta Gorda Street. Geotechnical engineering properties were assigned to various soil layers within the profile for use in evaluating the axial and lateral load capacity of pile foundations for bridge. For the purpose of evaluating the axial and lateral load capacity of the bridge we have assumed that the pile cap elevation (top of piles) will be approximately 2 feet below the bottom of the stream channel (approximate el. 8 feet).

Tables 2a and 2b provide our generalized soil profiles and summarize the thickness, description, and engineering properties assigned to selected layers used to characterize the subsurface conditions at the bridge abutments. Geologic boundaries were estimated based on CPT soundings and boring logs. Soil unit weights and strength parameters were estimated based on laboratory tests, the measured blow counts, correlation with the CPT soundings, and engineering judgment.

Table 2a. Engineering Parameters Abutment 1 (Northeast)

Elevation Interval (feet)	Generalized Soil Material	L-Pile Soil Type	Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle (degrees)	ε_{50} (in/in)	k (pci)
8 to -2	Medium stiff clay	Clay below water table	58 (buoyant)	750	--	0.020	70
-2 to -15	Stratified medium stiff clay and medium dense sand	Model as clay below water table	58 (buoyant)	750	--	0.020	70
-15 to -23	Medium dense sand	Sand (Reese)	58 (buoyant)	--	34	--	60
--23 to -35	Very dense sand	Sand (Reese)	58 (buoyant)	--	36	--	125

-35 to -58	Very stiff to hard clay	clay below water table	58 (buoyant)	3,000	--	0.005	1000
-58 to -83	Dense to very dense sand	Sand (Reese)	58 (buoyant)	--	36	--	125

Table 2b. Engineering Parameters Abutment 2 (Southwest)

Elevation Interval (feet)	Generalized Soil Material	L-Pile Soil Type	Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle (degrees)	ε_{50} (in/in)	k (pci)
8 to 6	Loose sand	Sand (Reese)	58 (buoyant)	--	32	--	20
6 to 1	Medium stiff clay	Clay below water table	58 (buoyant)	750	--	0.020	70
1 to -10	Stratified medium stiff clay and medium dense sand	Model as clay below water table	58 (buoyant)	750	--	0.020	70
-10 to -26	Medium dense sand	Sand (Reese)	58 (buoyant)	--	34	--	60
-26 to -34	Medium stiff clay	clay below water table	58 (buoyant)	1000	--	0.020	100
-34 to -42	Medium dense to dense sand	Sand (Reese)	58 (buoyant)	--	36	--	90
-42 to -58	Very stiff to hard clay	clay below water table	58 (buoyant)	3,000	--	0.005	1000
-58 to -83	Dense to very dense sand	Sand (Reese)	58 (buoyant)	--	36	--	125

5.0 SEISMIC HAZARD ASSESSMENT

5.1 SEISMIC SETTING

The project site is in a seismically active region of southern California. We performed a search of controlling faults in the area in accordance with current Caltrans Seismic Design Criteria and utilizing Caltrans ARS Online (Caltrans 2009a) and the 2007 Caltrans Deterministic PGA Map. Caltrans ARS Online is a web-based tool operated through the Caltrans website and is based on the Caltrans 2007 Fault Database that is continuously updated. ARS online displays information for faults included in the Caltrans 2007 Fault Database and calculates both deterministic and probabilistic acceleration response spectra (ARS) for any location in California as described in Appendix B of the Caltrans Seismic Design Criteria (Caltrans 2009b). ARS Online was first used to identify potential controlling faults in the site vicinity. Table 3 – Potential Controlling Faults presents a list of potential controlling faults closest to the site identified using ARS Online and site coordinates corresponding to Latitude 34.4211 and Longitude 119.6704. We also used ARS Online to estimate strong ground motion and develop a design ARS for the site as discussed in subsequent sections of this report.

Table 3. Potential Controlling Faults

Fault Name	Fault Type[1]	R_x Distance (mi)[2]	Maximum Magnitude (MMax)[3]
Mesa Rincon Creek Fault	Reverse	0.8	6.8
San Jose Fault (Santa Barbara)	Reverse	1.1	6.3
North Channel Slope Fault	Reverse	1.5	7.4
Mission Ridge- Arroyo Parida Fault	Reverse	1.8	7.2
More Ranch Fault	Reverse	2.2	7.2
Red Mountain Fault	Reverse	4.1	6.4

1): Fault type per Caltrans 2007 Fault Database.
2): Horizontal distance to the fault trace (fictitious fault trace for sites offset from the fault) or surface projection of the top of rupture plane measured perpendicular to the fault from the site per ARS Online and Caltrans Geotechnical Services Design Manual Version 1.0.
3). MMax values per ARS Online and Caltrans 2007 Database.

Brief descriptions of potentially controlling faults identified by ARS Online closest to the site are provided below.

Mesa Rincon Creek Fault. The Mesa Rincon Creek Fault identified on ARS Online is mapped south of the project site and dips to the south at 45 degrees. The site is located on the footwall of the fault.

San Jose Fault. The San Jose Fault identified on ARS Online is mapped northwest of the project site and dips to the south at 45 degrees. The site is offset from the fault and is located on the footwall of the fault.

North Channel Slope Fault. The North Channel Slope Fault identified on ARS Online is mapped south of the project site and dips to the northeast at 26 degrees. The site is located on the hanging wall of the fault.

Mission Ridge – Arroyo Parida Fault. The Mission Ridge – Arroyo Parida Fault identified on ARS Online is mapped north of the project site and dips to the south at 70 degrees. The site is is located on the hanging wall of the fault.

5.2 STRONG GROUND SHAKING

In accordance with the Caltrans Seismic Design Criteria, we used ARS Online to estimate strong ground motion and develop a design acceleration response spectra (ARS) for the project site. As discussed previously, ARS Online calculates both deterministic and probabilistic ARS for any location in California based on the Caltrans Seismic Design Criteria for faults included in the Caltrans 2007 Fault Database. Caltrans seismic design procedures include a comparison of the ARS Online estimated probabilistic ARS with the 2008 USGS Interactive Deaggregation Tool (Beta) (USGS, 2008) when the estimated shear wave velocity V_{s30} for the site is less than or equal to 300 meters/second. The development of design ARS for the site is discussed in Section 5.4.1 of this report.

Based on results of Caltrans seismic design procedures using ARS Online and comparison with results generated by the 2008 USGS Interactive Deaggregation Tool (Beta), a maximum considered (975-year return period) peak ground acceleration of 0.64g is estimated for the site. The Mission Ridge – Arroyo Parida Fault is the controlling fault for the deterministic ARS.

5.3 GROUND SURFACE RUPTURE

The site is not located in an Alquist-Priolo Earthquake Fault Zone as defined by the State of California. The closest significant faults to the project site identified in ARS Online are the Mesa Rincon Creek, San Jose, North Channel Slope, and Mission Ridge – Arroyo Parida Faults located approximately 0.8, 1.1, 1.5, and 1.8, miles from the site, respectively. On the basis of that information, in our opinion, the potential for ground surface rupture from faulting is considered to be low.

5.4 SEISMIC DESIGN CRITERIA

5.4.1 Design Response Spectra

A design acceleration response spectrum (ARS) curve for the site was developed using ARS Online and the requirements set forth in Appendix B of the Caltrans Seismic Design Criteria. The Caltrans Seismic Design Criteria also requires use of the 2008 USGS Interactive Deaggregation calculator (Beta version) as a tool during the development of the design probabilistic ARS curve when the estimated shear wave velocity V_{s30} for the site is less than or equal to 300 meters/second. We used CPT data and information on the boring logs performed for this study near the bridge site. We estimated shear wave velocities for materials encountered in the CPT soundings and borings by using correlations to CPT tip resistance, blowcount, and undrained shear strength and shear wave velocity presented in the Caltrans Geotechnical Services Design Manual (Caltrans, 2009c). An average shear wave velocity of 690 feet/sec was estimated for the top 100 feet of soil at the site. According to Appendix B of the Caltrans Seismic Design Criteria, a site with a shear wave velocity V_{s100} (V_{s30} in metric units) of 690 feet/sec (210 meters/second in metric units) corresponds to a Soil Profile Type D.

The deterministic and probabilistic spectra resulting from the ARS Online analysis for 5 percent damping are shown on Plate 3. The design deterministic ARS curve was controlled by the Mission Ridge – Arroyo Parida Fault. In accordance with Caltrans guidelines, we compared the site-specific deterministic ARS curve to the minimum deterministic ARS curve for California (defined by Caltrans as magnitude 6.5 vertical strike-slip event occurring at 7.5 miles from the site). The site-specific deterministic ARS curve is higher than the Caltrans minimum deterministic ARS curve for California for all periods. In accordance with Caltrans guidelines, the design ARS curve is taken as the upper envelope of the deterministic and probabilistic ARS curves. The design ARS for the project site is controlled by the probabilistic spectrum shown on Plate 3 with an estimated peak ground acceleration of 0.64g.

5.5 LIQUEFACTION

Liquefaction and seismic settlement hazards were evaluated for the site considering the design earthquake with a ground acceleration of 0.64g and a corresponding earthquake of M7.2. Liquefaction is a loss of soil strength due to a rapid increase in pore water pressures due to cyclic loading during a seismic event. Liquefaction potential of the soils was performed with the CPT data using procedures described in the 1997 NCEER guidelines (Youd and Idriss, 2001). Liquefaction commonly occurs in loose to medium dense sandy soil that is below the groundwater table at the time of an earthquake. The potential and severity of liquefaction will depend on the intensity and duration of the strong ground motion. Seismically induced settlement, ground deformation, lateral spreading, and loss of bearing support can occur as a result of liquefaction. Seismic settlement can occur in soils not prone to liquefaction (such as soils above the water level), that are very loose to medium dense and weakly cemented.

Groundwater water was encountered in the area of the proposed improvements as shallow as about 5 feet below the existing ground surface during our April 2010 field exploration. This depth appeared to correspond to the water level observed in Sycamore Creek during out field exploration. A groundwater depth of 5 feet below the existing ground surface was assumed for the liquefaction analyses. The fine-grained alluvial soils encountered in the explorations consisting of medium stiff to very stiff silt and clay are not considered susceptible to liquefaction. In general, the layers of loose to medium dense sandy alluvial soils encountered within the upper approximately 30 feet of soil material at the site are considered susceptible to liquefaction and prone to moderate seismic settlement. The results of our analyses are provided in Appendix E – Liquefaction Evaluation.

Based on analysis of the CPT soundings, we estimate that approximately 2 to 4 inches of seismic settlement could occur within the alluvial soils encountered at the site in response to the design earthquake. Because the soil profile is variable, the estimated settlement would likely occur as differential settlement across the site.

In general, the layers of granular alluvium that are considered susceptible to liquefaction appear to be relatively thin and/or discontinuous across the site, suggesting that the potential for lateral spreading is relatively low. However, some lateral deformation of the ground immediately adjacent to the creek could occur during a strong seismic event.

On the basis of the CPT data, the potential for liquefaction at the Punta Gorda Street bridge appears to range from about 23 feet (elevation -5 feet) at CPT-1 to about 28 feet (elevation -10 feet) at CPT-3. The analyses suggest that some liquefaction could occur below that depth. However, in our opinion, the potential for those layers to liquefy and result in ground deformation, settlement or impacts to deep foundations is considered unlikely. Potential downdrag loads from liquefaction were considered in the foundation design recommendations.

6.0 CONCLUSIONS

Our conclusions and recommendations are based on the exploration and testing programs described above, and on our understanding of the project. A geotechnical evaluation of the existing bridge piles are presented below.

6.1 SUMMARY OF FINDINGS

- A relatively thin layer (4 to 10 feet) of artificial fill materials appears to be present in the area adjacent to the bridge and along the banks of Sycamore Creek. The fill materials are underlain by alluvial strata of medium stiff to very stiff fine grained soil and medium dense to very dense granular soil. The thicknesses of the various strata range from about 1 foot to about 20 feet.
- Groundwater was encountered at depths of about 5 to 8 feet below the ground surface. For the purposes of our liquefaction and foundation analyses, groundwater was assumed to be present at a depth of about 5 feet below the ground surface (approximately elevation +15 feet).
- Based on our review of the as-built plans, the Punta Gorda Street bridge is supported on a gravity-type abutment wall with 3-foot-wide footings embedded below the creek flowline.
- We anticipate the new bridge will be supported on deep foundation elements consisting of cast-in-drill hole piles or driven low displacement piles (such as cast in steel shell piles/concrete-filled pipe piles) deriving resistance from the medium dense to dense sand and stiff to very stiff clay soils present below a depth of about 30 feet below the ground surface.
- The selection of the pile type used for the project will need to consider the potential difficulties associated with constructing CIDH piles using wet methods in stratified granular and fine-grained soils and the potential noise and vibration effects associated with driving low-displacement pipe piles at the site.

6.2 BRIDGE FOUNDATION EVALUATION AND DESIGN

In our opinion, shallow foundations are not considered suitable for support of the new bridge considering the presence of compressible, low to moderate strength clayey soils at the site and considering the potential consequences of liquefaction-related settlement. As a result, in our opinion, the new bridge should be founded on deep foundation elements that derive resistance from the medium dense to very dense granular soils and stiff to very stiff fine-grained soils present at the site below about elevation 10 feet.

For this project, we have evaluated both cast-in-drilled-hole (CIDH) piles and low-displacement steel pipe piles for support of the new bridge at Punta Gorda Street. Driven displacement piles (such as concrete piles or closed-end pipe piles) could also be used but vibration effects from driving displacements piles would likely be greater than compared to lower displacement piles. CIDH piles offer an advantage in terms of noise and vibration from driving; however, construction below the groundwater in the medium stiff clay and silt and loose to medium dense sand of the alluvium will be difficult and will require wet methods of installation. For the purposes of this report, we evaluated the axial and lateral capacities of a 30-inch CIDH pile and 24-inch-diameter x 0.625-inch pipe pile. Alternative pile types can also be evaluated on a specific, as-requested basis from the project team.

6.2.1 Axial Capacity

As discussed, in our opinion, it is feasible to support the proposed bridge on cast-in-drill-hole concrete piles. For this design alternative, we evaluated the axial capacity of a 30-inch-diameter cast-in-drill hole pile at abutments 1 and 2 assuming a pile cutoff elevation of +8 feet (about 2 feet below the creek bottom). The nominal resistance (ultimate capacity) of a 30-inch CIDH pile was evaluated using the procedures presented in Federal Highway Administration (1999) together with the computer program SHAFT (ENSOFT 2009). The nominal resistance of the pile was estimated based on the frictional resistance of the pile bearing in alluvium.

In our opinion, the proposed bridge can also be supported on driven piles provided the potential temporary noise and vibration impacts to the neighborhood are acceptable. Several pile types could be used for the project ranging from steel H or pipe piles to prestressed concrete. As indicated above, we have provided recommendations for a 24-inch-diameter pipe pile because of the potential advantages of high lateral capacity, low displacement during driving, flexibility in pile cut off, and the ability to perform center relief drilling if difficult driving conditions are encountered. Recommendations for other pile types or for pile lengths/capacities can be provided as needed.

The axial capacity for the 24-inch pipe pile was estimated for the assumed soil profile conditions at Abutments 1 and 2 using the program version 5 of the computer program APILE (Ensoft 2007a). The analyses were performed using the API method. End bearing for the pipe pile was assumed to be the lesser of the resistance provided by the internal soil plug (i.e. pile cores during driving) or end bearing of the full cross sectional area (i.e. pipe pile plugs during driving).

In accordance with the California Amendments to the AASHTO LRFD Bridge Design Specifications (Caltrans 2009d), LRFD is not required for the design of abutment or retaining wall foundations. Therefore, design capacities were estimated using allowable stress design methods and a factor of safety of 2 for the CIDH and driven piles.

The granular soils layers in the upper 30 feet (or to an elevation of about 10 feet) are considered susceptible to liquefaction causing the soils above a depth of about 30 feet to settle and adding downdrag loads to the piles. Estimates of the downdrag load caused by liquefaction are provided in Table 4 – Pile Design Summary for Axial Loading. Longer piles or alternative pile types could be evaluated for final design if needed.

Table 4. Pile Design Summary for Axial Loading

Location	Estimated Downdrag Load, Unfactored tons	Nominal Resistance (Ultimate Capacity), Including Unfactored Downdrag Load tons	Design Loading (Working Stress Design) tons	Design Tip Elevation for Compression Loading (ft)[1]
Abutment 1 30-inch CIDH	12	92	40	-30
Abutment 2 30-inch CIDH	24	104	40	-37
Abutment 1 24-inch Pipe Pile	12	152	70	-42
Abutment 2 24-inch Pipe Pile	25	165	70	-49

1 – Based on a cutoff elevation of +8 feet

6.2.2 Lateral Pile Capacity

Lateral pile load carrying capacity was estimated using the computer program LPILE Plus 5.0 (Ensoft 2004) with a soil resistance-pile deflection model (p-y analysis). LPILE was used to estimate lateral load deflection and maximum moment for the piles for a range of lateral loads at the pile head. Both fixed- and free-head conditions were evaluated.

Table 5. Lateral Pile Capacity

Pile Type	Pile Head Fixity	Lateral Load (kips)	Approximate Pile Head Displacement (in)	Maximum Bending Moment (ft-kips)	Estimated Critical Pile Length/Elevation* (ft)
30-inch CIDH	Free	30	0.25	196	35 / -27
30-inch CIDH	Fixed	55	0.25	438	35 / -27
24-inch steel pipe	Free	23	0.25	136	30 / -22
24-inch-steel pipe	Fixed	50	0.25	393	35 / -27

*Defined per Caltrans Memo to Designers 3-1 July, 2008.

Soil conditions assumed for the analyses were limited to the profile information for Abutment 1 because the conditions were considered sufficiently similar between Abutments 1 and 2

No factors of safety (a resistance factor of 1.0) were applied to the estimated loads or deflections. An axial load in the pile of 50 tons was assumed in the lateral analyses. Group effects were not considered in the analyses but can result from shadowing of adjacent piles.

Fugro should provide input for group effects for lateral loading after the pile layout has been determined.

6.2.3 Settlement

We estimate that settlement for an isolated pile should not exceed one inch under the 50 ton allowable design load. Differential settlements between the abutments can be estimated as about half of the estimated total settlement.

6.2.4 Pile Data Table

The recommended pile data table for the two pile types considered for the project is provided below.

Table 6. Pile Data Table

Support Location and Pile Type	Estimated Downdrag Load, Unfactored tons	Nominal Resistance tons	Design Loading (Working Stress Design) tons	Design Tip Elevation[1,2] ft	Specified Tip Elevation[1] ft	Nominal Driving Resistance Required[3] tons
Abutment 1 30-inch CIDH	12	80 Compression 0 Tension	40	-30 (a,c) -27 (b)	-30	NA
Abutment 2 30-inch CIDH	24	80 Compression 0 Tension	40	-37 (a,c) 27 (b)	-37	NA
Abutment 1 24-inch Pipe Pile	12	140 Compression 0 Tension	70	-42 (a,c) -22 (b)	-42	152
Abutment 2 24-inch Pipe Pile	25	140 Compression 0 Tension	70	-49 (a,c) -27 (b)	-49	165

1 – Based on a cutoff elevation of +8 feet
2 - Design tip elevations are controlled by: (a) compression, (b) lateral load, (c) liquefaction, (d) settlement
3 - Liquefiable soils are present at Abutment 1 and Abutment 2 to approximately elevation -5 and -10 respectively that do not contribute to the nominal resistance

The data provided in Table 6 – Pile Data Table, are considered applicable to wingwall foundations if those types of structures will be included in the design.

6.3 RETAINING WALLS

Abutments and wingwalls should be designed according to the recommendation of this report. We recommend the following equivalent fluid weights for use in estimating the lateral earth pressures that will act on retaining walls with level backfill conditions and active earth pressure conditions. Retaining wall backfill should consist of granular soil materials meeting Caltrans Standard Specifications for Structure Backfill.

Table 7. Lateral Earth Pressures

Lateral Earth Pressure Distribution	Backfill Material	Equivalent Fluid Weight (pcf)
Active – Unbraced Drained	Structure Backfill	35
Active – Unbraced Undrained	Structure Backfill	80
At Rest – Braced Drained	Structure Backfill	55
At Rest – Braced Undrained	Structure Backfill	90

Drained values do not provide for hydrostatic forces (for example, standing water in the backfill material). If drainage cannot be provided behind the walls, undrained conditions should be assumed and used for design. Surcharge stresses from vehicle traffic can be estimated as a uniform surface load of 250 psf resulting in a uniform pressure on the wall of about 100 psf. If conditions (other than surcharge resulting from traffic loads) are anticipated, Fugro should be advised so we can provide additional recommendations as needed.

As discussed above, for drained backfill conditions, drainage should be provided behind retaining walls to reduce the potential for the buildup of hydrostatic pressures. Retaining walls designed for drained loading conditions should be designed with weep holes or collector pipes to assist in the removal of water from the backfill, and to prevent the build up of hydrostatic pressures behind the wall.

Structure backfill should be placed between the wall and a 1h:1v backslope projected up from the heel of the footing. If the design of the wall assumes no hydrostatic pressures (drained condition) acting on the wall, a continuous layer of drainage material consisting of either 1-foot of drainage material, or Geocomposite Drain panels should be provided along the backside of the wall. The drainage material should be terminated 2 feet below the finished grade of the wall backfill, and be topped with on-site soil or topsoil.

6.4 CORROSION CONSIDERATIONS

A selected sample of the native alluvial soils was obtained from DH-1 and was tested for pH, resistivity, and chloride and sulfate content. The results of the test are provided in Appendix B and outlined below in Table 8 - Summary of Corrosion Test Results.

Table 8. Summary of Corrosion Test Results

Drill Hole No.	Sample Depth (ft)	Material Type	Resistivity (ohms/cm)	PH	Chloride (ppm)	Sulfate (wt %)
DH-1	30.5	Silty Sand	3089	8.3	15	<0.0005 (<5 ppm)

The above corrosion data suggest that the alluvial deposits at the site do not meet Caltrans' criteria (Caltrans 2003) for a corrosive environment and therefore the potential for corrosion appears to be low. However, steel piles should be provided with additional thickness (sacrificial steel) in accordance with Caltrans requirements.

6.5 ASPHALT PAVEMENT DESIGN RECOMMENDATIONS

We anticipate that some limited pavement improvements will be required for the project. On the basis of the exploration data acquired for the site, in our opinion, it is likely that subgrade conditions for pavement improvements will likely consist of fine-grained soil material. As a result, we have assumed an R-value of 10 for use in developing recommendations for the preliminary pavement structural sections for the project. Pavement structural sections were estimated using the methods described in Caltrans Highway Design Manual (2009e) and are summarized in Table 9 – Preliminary Pavement Structural Sections for a range of traffic loading conditions. Final design sections should be based on specific R-value testing of the actual subgrade soils collected during rough grading.

Table 9. Preliminary Pavement Structural Sections: R-Value = 10

Traffic Index	Thickness of Asphalt Concrete	Thickness of Aggregate Base
7	0.35 feet	1.15 feet
8	0.40 feet	1.35 feet
9	0.45 feet	1.6 feet

We recommend that the upper 1-foot of subgrade and aggregate base material be compacted to a minimum of 95 percent relative compaction as determined by the latest approved edition of ASTM Test Method D 1557.

6.6 CREEK SEDIMENT SAMPLING

We collected six grab samples of sediment from the creek channel bottom during our field exploration in April 2010 as shown on Plate 2. Water was flowing in the creek during sediment sampling. We performed grain size analyses (sieve and hydrometer tests) on each sample. The results of grain size analyses on the sediment samples are shown in Appendix B (Plate B-3b). The sediment samples tested consisted of poorly-graded sand (SP) and poorly-graded sand with silt (SP-SM).

6.7 SYCAMORE CREEK SLOPE STABILITY EVALUATION

We understand that the proposed channel improvements to Sycamore Creek will include widening the existing channel to 60 feet at the top of the bank with 2h:1v natural soil side slopes. We performed slope stability analyses to evaluate the proposed channel geometry considering the subsurface conditions encountered during our field exploration. The ground surface profile was estimated using the existing creek channel conditions, dimensions for the proposed channel geometry and topography provided by Penfield & Smith (2010). Based on the proposed channel geometry and the existing width of the creek, the proposed slope will be approximately 12 feet high. Slope stability analyses were performed for static loading, pseudostatic (earthquake) loading, and rapid drawdown conditions. The slopes were evaluated with respect to the stability criteria discussed below. Computer output and results from the slope stability analyses can be provided upon request.

6.7.1 Slope Stability Criteria and Method of Analysis

Slope stability criteria were selected in accordance with the State's Guidelines for Evaluating and Mitigating Seismic Hazards (CDMG 1997). For the purpose of evaluating analytical results, slopes are considered stable when the estimated factor of safety is at least 1.5 under static loading conditions and at least 1.1 under pseudostatic (earthquake) loading conditions when using a horizontal pseudostatic coefficient of 0.15. A factor of safety 1.0 represents the theoretical boundary below which a slope is no longer stable and experiences failure. Factors of safety greater than 1.0 are theoretically stable; however, a factor of safety of at least 1.5 is typically used to define stable slope conditions in practice to help account for uncertainties associated with characterizing subsurface conditions and limitations associated with the geotechnical analyses used to evaluate slope stability.

The slope stability analyses were performed using the computer program GSTABL7 with STEDwin, Version 2.005 (Gregory 2006). GSTABL7 was used to estimate factors of safety for slope stability under static and pseudostatic loading conditions. GSTABL7 requires the user to input the surface and subsurface profile boundaries; soil properties including unit weight (γ), friction angle (ϕ) and cohesion (c); groundwater levels; and the analysis method to be used.

6.7.2 Selection of Shear Strength Parameters

Effective shear strength parameters (ϕ and c) were selected for slope stability analyses based on interpretations from CPT soundings and borings performed along the existing channel at the top of the bank and the results of laboratory direct shear tests. The soil materials and strength parameters used for the analyses consisted of the following:

Soil Material	**Depth Interval Relative to the Slope Crest**	**Cohesion**	**Friction Angle**
Silty Sand	0 to 5 feet	0 psf	33 degrees
Sandy Clay	5 to 15 feet	100 psf	30 degrees

Groundwater was encountered in the borings and CPT soundings performed along the existing creek channel at depths ranging between approximately 5 and 8 feet below the existing ground surface. We considered the groundwater conditions along the existing channel both northwest and southeast of the Punta Gorda Street Bridge in our analysis.

6.7.3 Summary of Slope Stability Results

The slope stability analyses were performed to estimate the minimum static and pseudostatic factors of safety for the proposed channel geometry and the soil conditions encountered. The estimated factor of safety for the static and pseudostatic condition is approximately 1.9 and 1.3, respectively. The factors of safety are above the minimum 1.5 and 1.1 factors of safety used to define stable slope conditions for static and pseudostatic loading conditions. The estimated factor of safety for the rapid drawdown condition is approximately 1.5 for the static condition. Based on the result of our slope stability analyses and the soil conditions encountered during our field exploration, the proposed channel geometry should result in a stable slope condition.

6.8 CONSTRUCTION CONSIDERATIONS

6.8.1 General

The proposed channel improvements below the bridge are expected to consist of removing the existing bridge at Punta Gorda Street and regrading the Sycamore Creek channel from Highway 101 to about Soledad Street. Grading and other work will be performed on the creek banks and will likely extend into the creek bottom area as well. Temporary excavations, surface water control, and temporary dewatering will likely be required to construct the proposed bridge and make the proposed improvements to the creek channel.

6.8.2 Temporary Slopes

The contractor should be responsible for providing and maintaining safe excavations in accordance with State of California, Occupational Safety and Health Administration (CAL/OSHA) regulations. The contractor should continuously monitor temporary slopes. As a general guide, we recommend that temporary excavations that are properly dewatered (that is free of groundwater seepage from the slope) and less than about 20 feet high be inclined no steeper than 1h:1-1/2v. However, slopes may have to be flattened if slope instability, caving, or groundwater seepage is observed.

We note that unsupported slopes are likely to yield as the ultimate shear strength of the slope materials is mobilized. The yielding materials could have detrimental effects on improvements on adjacent properties, particularly within a distance from the top of the slope equal to the slope height. We recommend that lateral excavation support be provided in areas where existing improvements are located within a horizontal distance of the channel wall equal to two times the channel wall height.

6.8.3 Temporary Excavation Support

Lateral excavation support is recommended in areas where adjacent structures do not allow temporary excavations at 1h:1-1/2v, and/or areas where lateral or vertical movements are not acceptable (i.e., areas where improvements lie within a horizontal distance less than two times the channel height from the channel wall). Temporary support may include sheetpiling, soldier piles with lagging and tiebacks (in accessible areas with no below-ground obstructions such as pools), or other measures.

Retained soils behind temporary flexible supports, such as sheetpiling, are prone to lateral and vertical movements, as the retained soil mobilizes shear strength for cantilever design, or to a lesser degree, for the at-rest (i.e., restrained) state. The amount and type of movement depends on the soil characteristics, wall type, and construction procedures.

The contractor should be responsible for the design of shoring systems such that the construction will not result in adverse settlement, lateral movement, or instability of improvements located on adjacent private and public property. Moreover, the designer of the temporary support system should estimate lateral deflections and implement design measures (e.g., bracing) that maintain deflections to levels acceptable to the County. Voids between the temporary shoring system and the backcut should be filled with non-compressible fill material. Additionally, movements should be monitored during construction as described above. Shoring plans and design should be submitted for review for conformance with the recommendations in this report.

We anticipate that potential shoring methods could consist of cantilevered or braced sheet piling or soldier beam and lagging systems. Lateral pressures applicable for the design will depend on the type of shoring system selected by the contractor, surcharge loads due to construction equipment and traffic, and any dewatering methods that are used.

6.8.4 Cast-In-Drill-Hole Piles

Cast-in-drilled-hole piles should be installed in general accordance with Section 49 of the Caltrans Standard Specifications and recommendations presented herein. We anticipate that drilled shafts excavated for the project will encounter interbedded or stratified coarse- and fine-grained alluvium with local zones potentially containing gravel. The soils are generally loose to medium dense/medium stiff to stiff above a depth of 30 feet and medium dense to very dense/very stiff to hard below about 30 feet. Groundwater will be encountered at relatively shallow depths.

We anticipate that excavations for drilled shafts will likely require the use of casing and/or drilling slurry to advance the drilled shafts to the design depth. The contractor should recognize the potential for groundwater, caving soils, and local gravel in the subsurface and provide the appropriate equipment to construct the CIDH piles. Project specifications should indicate that drill casing and drill slurry will be required. The contractor should provide a detailed submittal describing the proposed construction methods and sequencing of events, materials, type of slurry and procedures to monitor and test the condition of the slurry prior to concrete placement, equipment, and personnel. This submittal should be reviewed by the engineer prior to mobilization. Procedures for monitoring and testing the slurry should conform to Caltrans requirements. Inspection tubes for gamma-gamma and cross sonic logging (CSL) should be installed by the contractor and the contractor should provide for gamma-gamma and CSL testing and evaluation of the completed CIDH piles. Personnel employed to perform and interpret the non-destructive tests should have specific expertise in that type of work. Defects noted from the logging should be repaired by the contractor.

Construction methods should consider the proximity of the work area to Sycamore Creek and to potential environmental issues associated with constructing CIDH piles using casing and/or drilling slurry. The conditions of the drilled hole should be reviewed for stability before placing reinforcing steel and concrete. Loose soils should be removed with a flat bottom clean-out bucket prior to the placement of reinforcing steel.

Reinforcing steel should be provided with spacers to ensure the required spacing from the sides of the drilled hole. Concrete should be placed using a tremie pipe and concrete boom truck and have a slump of about 7 to 8 inches. The concrete should be tremied to displace groundwater (and loose soils) present at the bottom of the hole. Casing should be pulled as the concrete level rises in the drilled shaft however a minimum 8 feet of fresh concrete should be inside the casing at all times. The tremie pipe should remain in the fresh concrete through completion of concrete placement. Improper retrieval of the casing or reinsertion of the tremie pipe can cause necking or contamination of the concrete. As noted above, the contractor should take special precautions to contain water, mud, drilling slurry, and concrete in the work area.

6.8.5 Pile Driving Considerations

Piles should be driven and installed to the required penetration(s) in accordance with Section 49 of the Caltrans Standard Specifications (Caltrans 2006). The contractor should be responsible for selecting the equipment to be used for pile driving and for achieving the required penetration with the pile remaining in good condition after driving. In addition, the work should be performed in a manner that will not impact existing infrastructure or adjacent residential structures. We recommend that a pile driving criteria be established using wave equation methods by the contractor prior to mobilizing personnel and equipment to the site. The analysis should consider the pile type, embedment depth, and hammer-driving system to be used. .

Piles should not be terminated above the specified tip elevation unless pile-driving refusal is met. If refusal is met above the tip elevation, center relief drilling, predrilling, or other measures should be employed prior to driving subsequent piles such that refusal is met at the specified tip elevation.

Fugro should review the results of the contractor's pile driveability analyses and be contacted to provide observation and monitoring of the production pile driving activities.

6.8.6 Monitoring of Existing Structure

We recommend that shoring systems be designed and constructed in a manner to not cause damage to existing infrastructure or existing residences in the project area. We recommend a detailed inspection be made prior to construction of existing structures adjacent to the proposed temporary construction slopes. The inspection should include visual observations, photographs, and video documentation of the conditions of the existing structures. We also recommend that a monitoring program be performed during construction to evaluate the impact of the construction on the existing structures.

7.0 LIMITATIONS

This geotechnical study report has been prepared for Penfield & Smith Engineers solely for planning and design of the Sycamore Creek Enhancement project in Santa Barbara, California.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, or atmosphere, although samples for water quality testing were obtained and submitted for analysis. Any statements, or absence of statements, in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessments.

In performing our professional services, we have used generally accepted geologic and geotechnical engineering principles and have applied that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers currently practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report.

Results, evaluations, conclusions, and recommendations contained in this report are directed at and intended to be utilized within the scope of work contained in the proposal executed by Fugro and the client. This report is not intended to be used for any other purposes. Fugro makes no claim or representation concerning any activity or condition falling outside its specified purposes to which this report is directed, said purposes being specifically limited to the scope of work as defined in said agreement. Inquiries as to said scope of work or concerning any activity not specifically contained therein should be directed to Fugro for determination and, if necessary, further investigation.

We recommend that Fugro West, Inc., be retained to review and comment on geotechnical aspects of the project plans and specifications before they are finalized. This can allow Fugro West, Inc., to evaluate if the recommendations in this report have been properly interpreted and implemented in the design, specifications, and drawings.

Users of this report should recognize that the construction process is an integral design component with respect to the geotechnical aspects of the project. Because geotechnical engineering is inexact due to the variability of the natural processes, unanticipated or changed conditions can occur. Proper geotechnical observation and testing during construction is thus imperative in allowing the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, we recommend that Fugro West, Inc., be retained during site grading, excavation, and construction of foundations to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those anticipated.

8.0 REFERENCES

Caltrans (2003), Corrosion Guidelines, Version 1.0, dated September 2003.

Caltrans (2006), Standard Specifications, dated May 2006.

Caltrans (2009a), ARS Online Version 1.0.4, http://dap3.dot.ca.gov/shake_stable/

Caltrans (2009b), Appendix B – Seismic Design Criteria, dated September 11, 2009.

Caltrans (2009c), Geotechnical Services Design Manual Version 1.0, dated August 2009.

Caltrans (2009d), California Amendments (to the AASHTO LRFD Bridge Design Specification, V.4)

Caltrans (2009e), Highway Design Manual, Sixth Edition

Ensoft, Inc. (2004), "Lpile Plus Version 5.0," a program for analyzing the stress and deformation of a pile or drilled shaft under lateral loading.

Ensoft, Inc. (2007a), "APILE Plus Version 5.0," A program for the Analysis of the Axial Capacity of Driven Piles.

Ensoft, Inc. (2007b), "SHAFT Version 6.0," A program for the Study of Drilled Shafts Under Axial Loads.

Federal Highway Administration (FHWA, 1996), "Design and Construction of Driven Pile Foundations," FHWA Report No. FHWA-HI-97-013, dated December.

USGS (2008), Interactive Web Page for Seismic Design Values for Buildings, Version 5.0.9 – 10/6/08, http://earthquake.usgs.gov/research/hazmaps/design/index.php

Youd, T.L. and Idriss, I.M. (2001), "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Co-chairs Youd, T.L. and Idriss, I.M., Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, pp. 817-833.

PLATES

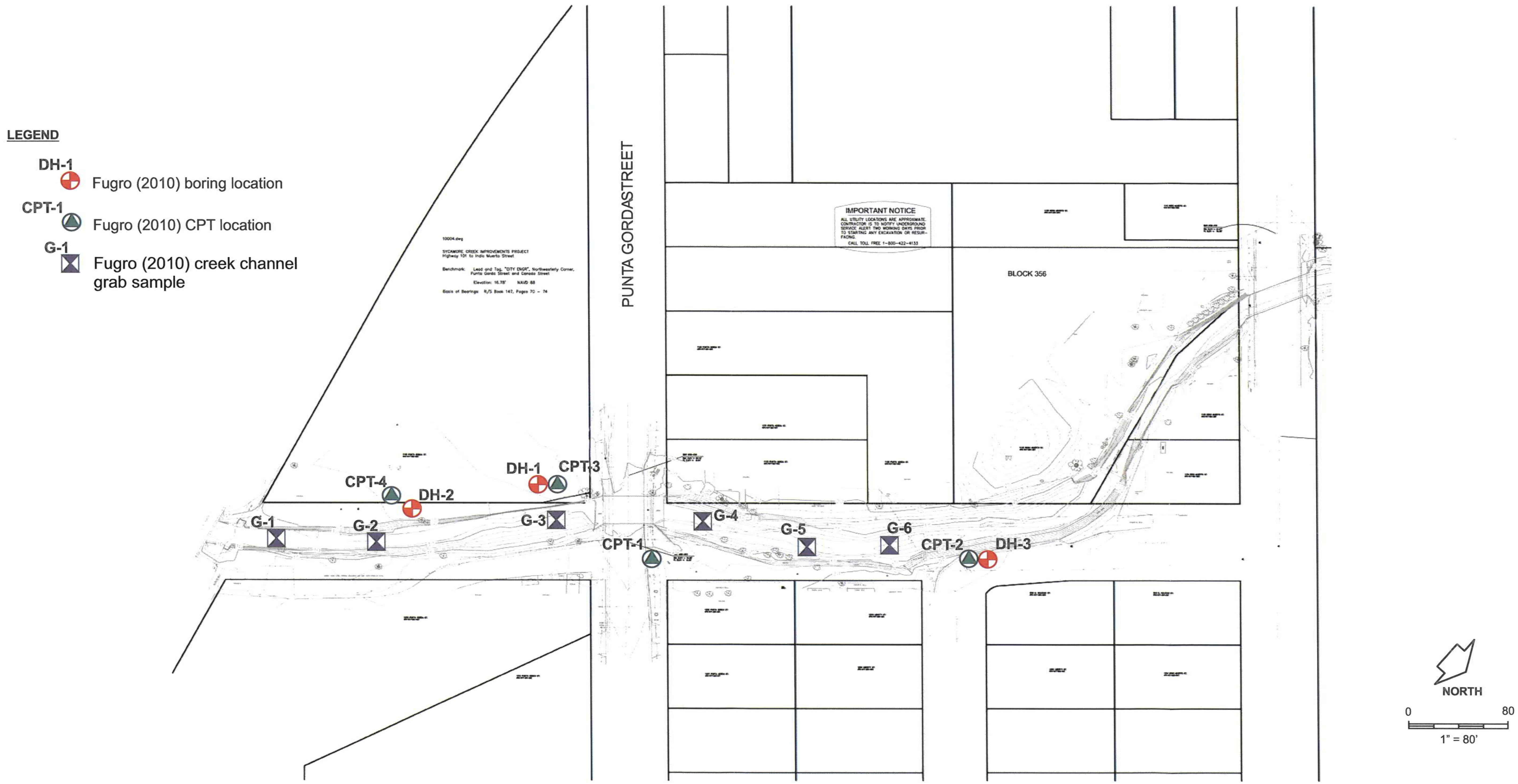
BASE MAP SOURCE: USGS 7.5' Santa Barbara 1995 Quadrangle.

NORTH
0 1000 2000
FEET

VICINITY MAP
Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE 1

Penfield & Smith
Project No. 3037.047

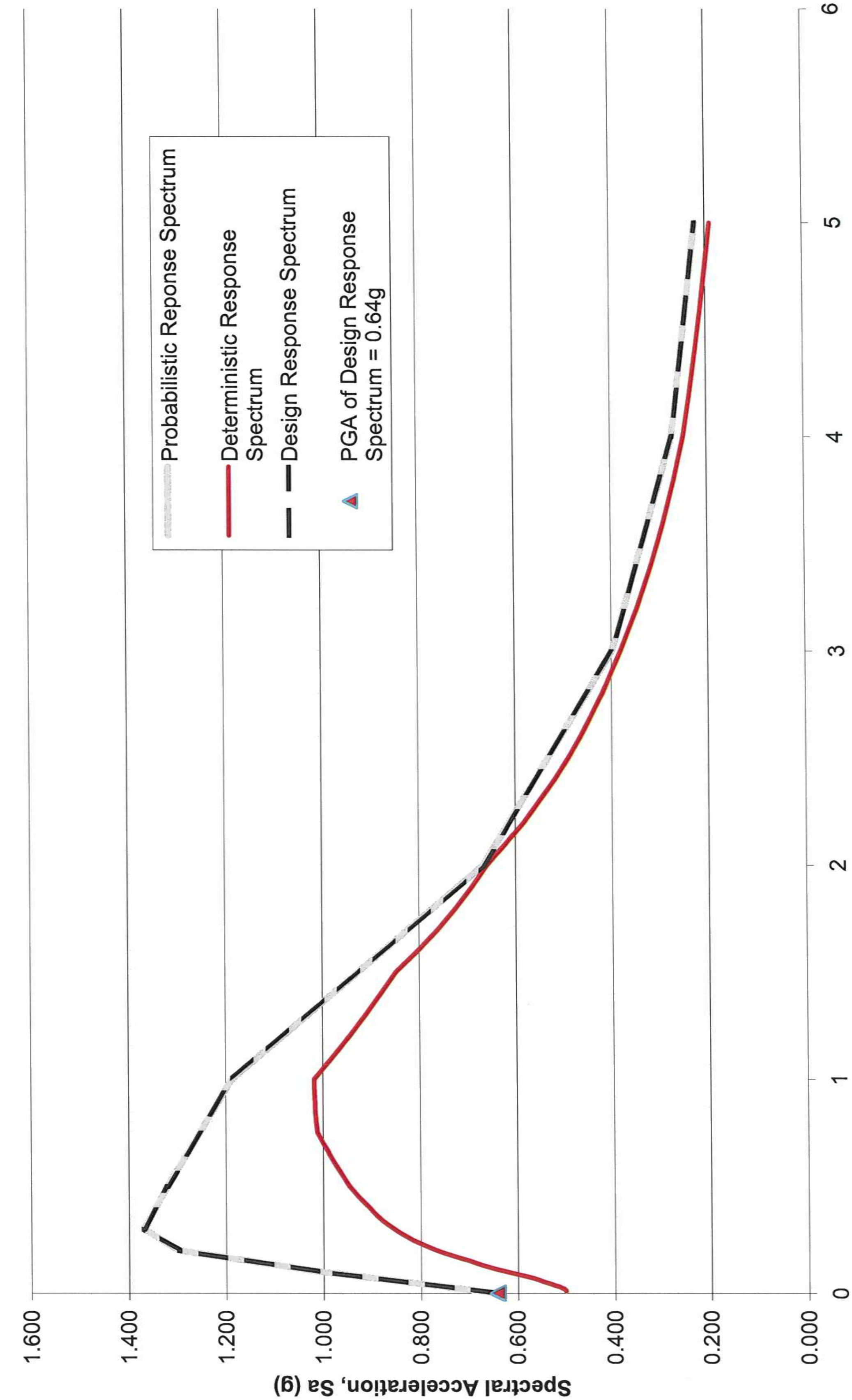


M:\Drafting\JOBFILES\2010\3037\3037.047\Drawings\B3037.047-02 site.cdr, 7-19-10

FIELD EXPLORATION PLAN
Sycamore Creek Enhancement Project
Santa Barbara, California

Base Map: Penfield & Smith Topographic Base Map (2010).

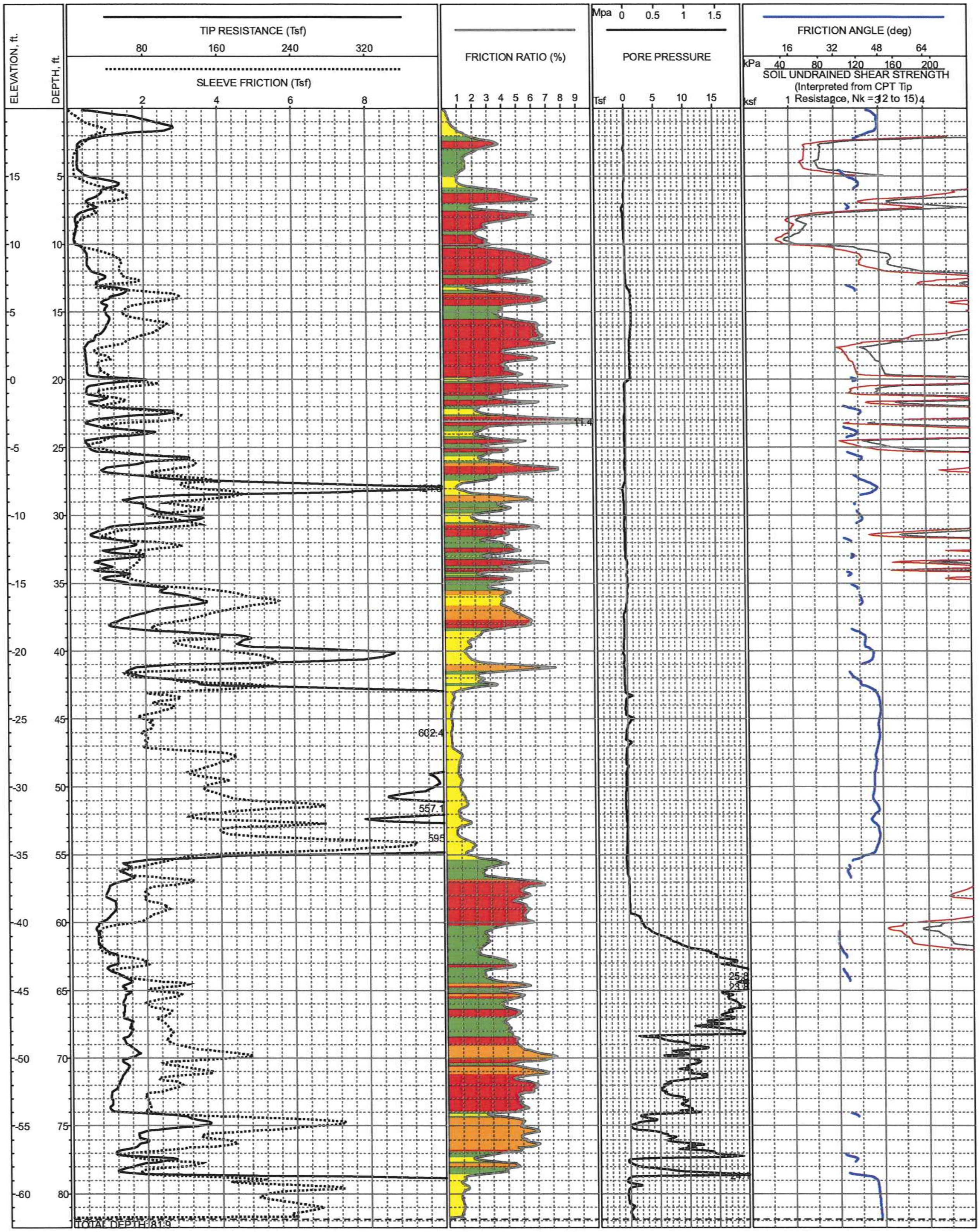
PLATE 2



DESIGN RESPONSE SPECTRA
Sycamore Creek Enhancement Project
Santa Barbara, California

APPENDIX A
SUBSURFACE EXPLORATION

Penfield & Smith Engineers
Project No. 3037.047



LOCATION:
SURFACE EL: 20.0ft +/- (MSL)
COMPLETION DEPTH: 81.9ft
TESTDATE: 3/31/2010

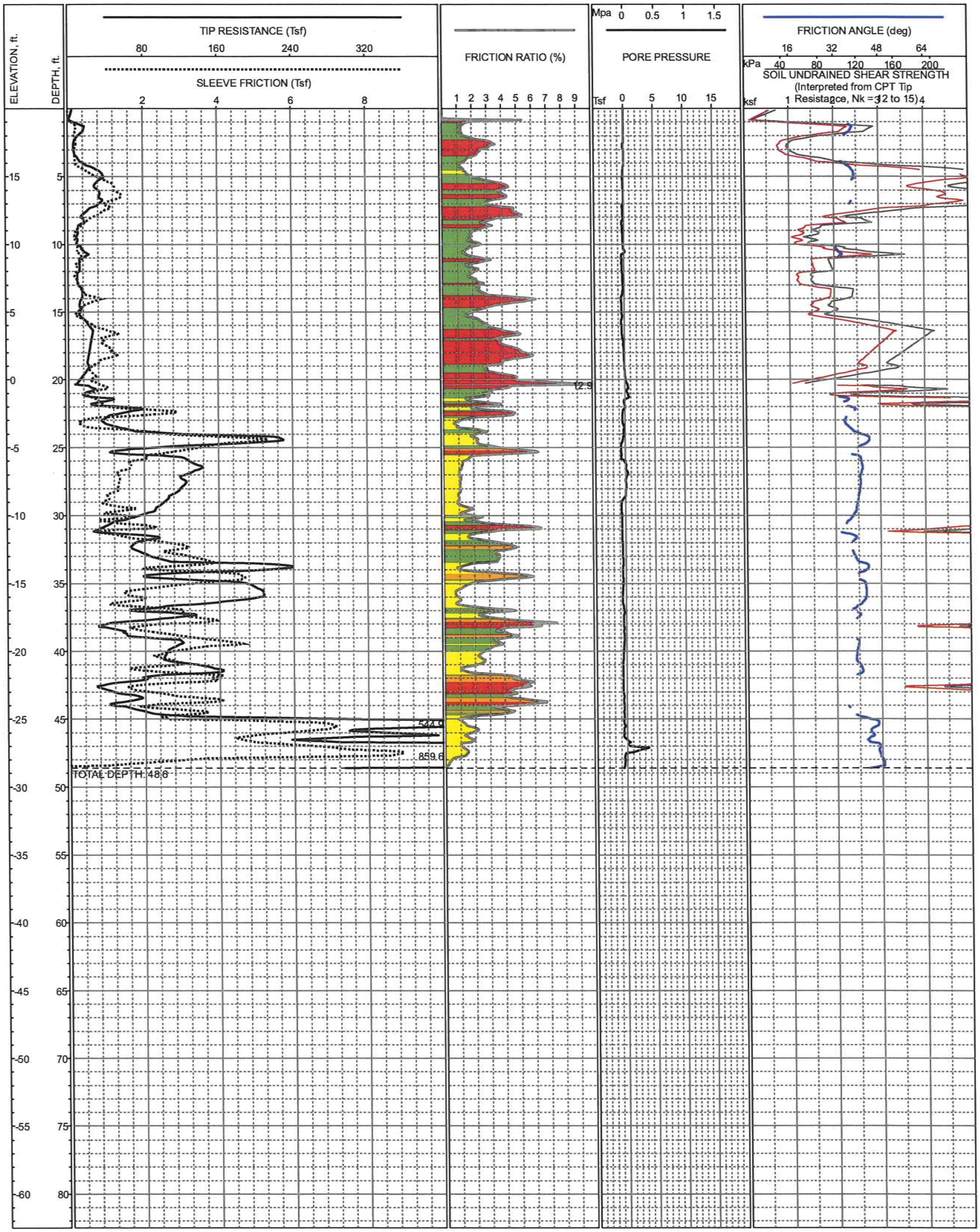
EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: K Robinson

LOG OF CPT NO: CPT-1

Sycamore Creek Enhancement Project
Santa Barbara, California

N:\Projects\3037_PenfieldSmith\3037-047_SycamoreCreek\Explorations\CPT\2010\Logs\2010_04_29_Logs_Su_Fr\MXD\Logs_VK10C_Su_Fr.mxd,04/29/2010,CDean

Penfield & Smith Engineers
Project No. 3037.047



LOCATION:
SURFACE EL: 20.0ft +/- (MSL)
COMPLETION DEPTH: 48.6ft
TESTDATE: 3/31/2010

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: K Robinson

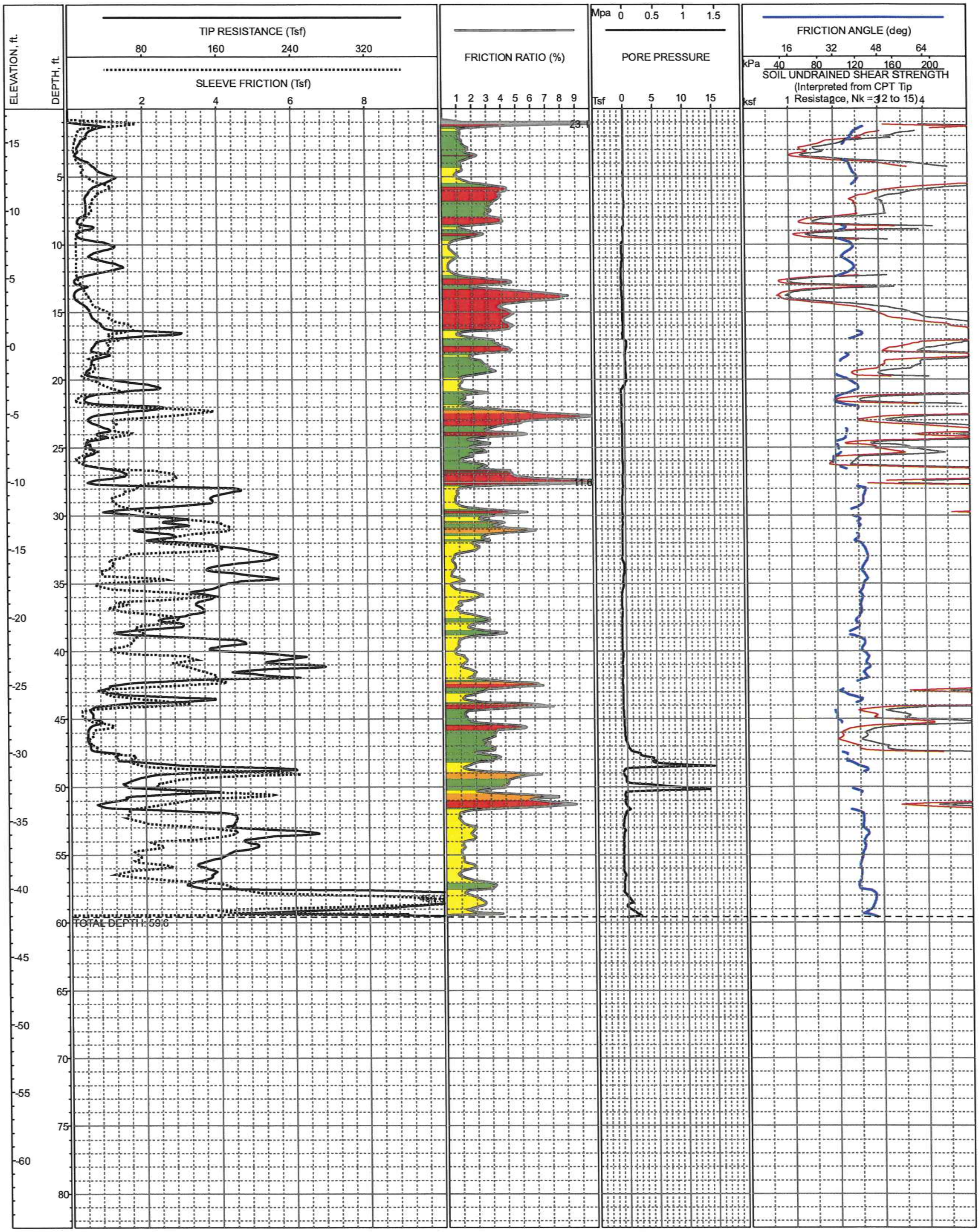
LOG OF CPT NO: CPT-2

Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE A2

N:\Projects\3037_PenfieldSmith\3037-047_SycamoreCreek\Explorations\CPT\2010\Logs\2010_04_29_Logs_Su_Fr\MXD\Logs_VK10C_Su_Fr.mxd,04/29/2010,CDean

Penfield & Smith Engineers
Project No. 3037.047



LOCATION:
SURFACE EL: 17.5ft +/- (MSL)
COMPLETION DEPTH: 59.6ft
TESTDATE: 4/1/2010

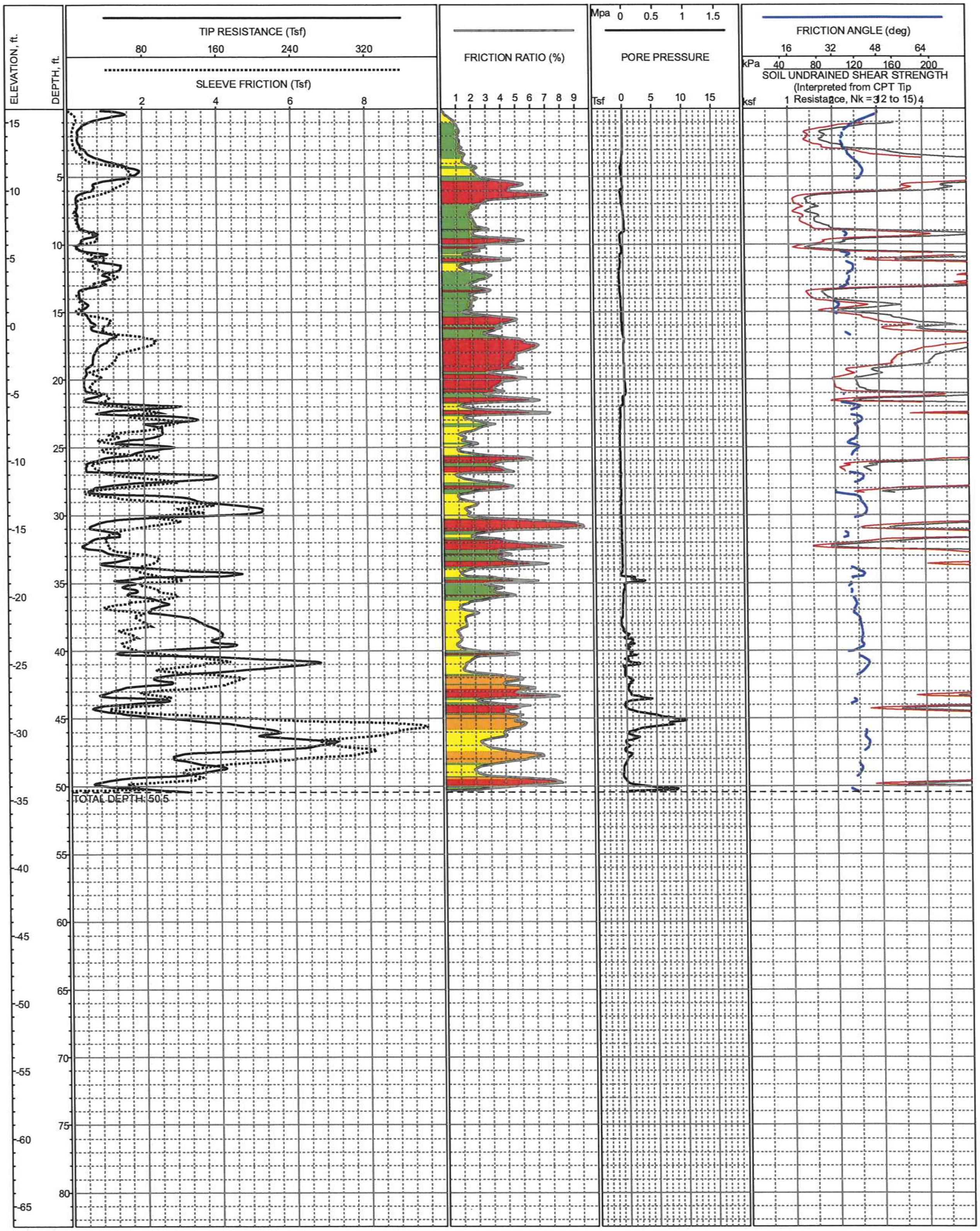
EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: K Robinson

LOG OF CPT NO: CPT-3

Sycamore Creek Enhancement Project
Santa Barbara, California

N:\Projects\3037_PenfieldSmith\3037-047_SycamoreCreek\Explorations\CPT\2010\Logs\2010_04_29_Logs_Su_Fr\MXD\Logs_VK10C_Su_Fr.mxd,04/29/2010,CDean

Penfield & Smith Engineers
Project No. 3037.047



LOCATION:
SURFACE EL: 16.0ft +/- (MSL)
COMPLETION DEPTH: 50.5ft
TESTDATE: 4/1/2010

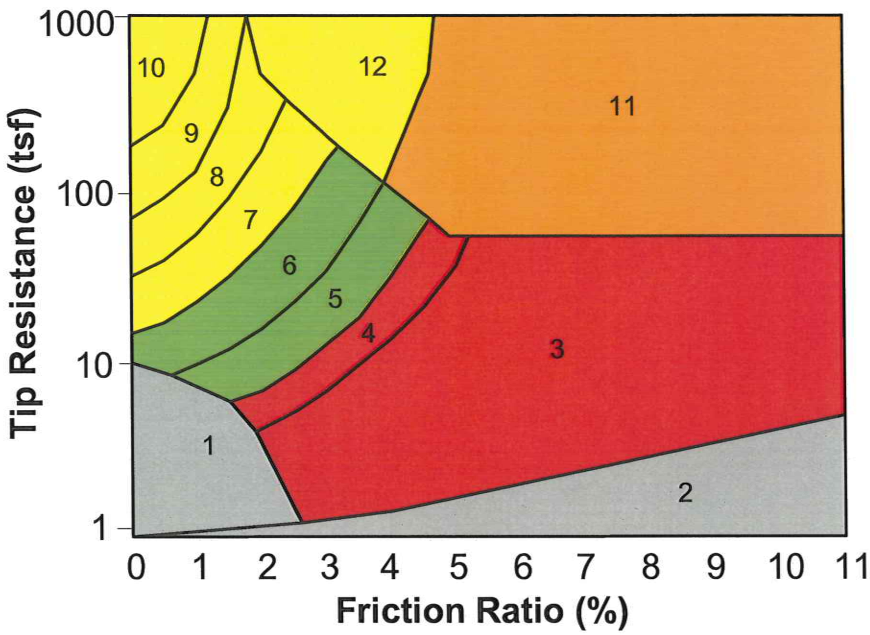
EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: K Robinson

LOG OF CPT NO: CPT-4

Sycamore Creek Enhancement Project
Santa Barbara, California

N:\Projects\3037_PenfieldSmith\3037-047_SycamoreCreek\Explorations\CPT\2010\Logs\2010_04_29_Logs_Su_Fr\MXD\Logs_VK10C_Su_Fr.mxd,04/29/2010,CDean

COLOR LEGEND FOR FRICTION RATIO TRACES



Zone	Soil Behavior Type	U.S.C.S.
1	Sensitive Fine-grained	OL-CH
2	Organic Material	OL-OH
3	Clay	CH
4	Silty Clay to Clay	CL-CH
5	Clayey Silt to Silty Clay	MH-CL
6	Sandy Silt to Clayey Silt	ML-MH
7	Silty Sand to Sandy Silt	SM-ML
8	Sand to Silty Sand	SM-SP
9	Sand	SW-SP
10	Gravelly Sand to Sand	SW-GW
11	Very Stiff Fine-grained *	CH-CL
12	Sand to Clayey Sand *	SC-SM

*overconsolidated or cemented

CPT CORRELATION CHART

(Robertson and Campanella, 1988)

N:\Projects\3037_PenfieldSmith\3037-047_SycamoreCreek\Explorations\CPT\2010\Logs\2010_04_29_Logs_Su_Fr\MXD\Plate_A5_KeytoCPT-RC88.mxd, 07/27/10, CDean

KEY TO CPT LOGS

Sycamore Creek Enhancement Project
Santa Barbara, California

LOCATION: See Plate 2 - Subsurface Exploration Location PlanN 1,979,586 E 6,058,062

SURFACE EL: 18 ft +/- (rel. NAVD 88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
		A		**ARTIFICIAL FILL (af)** Sandy Lean CLAY (CL): dark brown (7.5YR3/2), moist, fine sand, with roots and some fine gravel							
16	2										
14	4										
			(15)	- turns to brown (7.5YR4/2) with dark brown mottles, with yellow sand inclusions, piece of clay tile	129	112	15				
12	6			Silty CLAY (CL-ML) to Sandy Lean CLAY (CL) with sand and gravel lenses: stiff, brown (7.5YR4/2 to 4/3), moist, fine to medium sand							p 2.3
10	8			▽							
											t 0.6
8	10		(10)	- becomes medium stiff at ~9-1/2', dark gray (7.5YR4/1) silty clay, wet	126	101	25	61			
				- sand lens at ~10', silty fine sand grades to well-graded sand with gravel, fine to coarse sand, fine gravel		107	19				
6	12										p 2.5
4	14										
			(17)	- clayey sand lens at ~14-1/2', medium dense, fine to coarse sand, with some fine gravel, grades to sandy clay at ~15'	133	114	17	22			
2	16										
0	18										
-2	20			- fine gravel lense at ~20', subangular gravel							
-4	22										
-6	24										
			(13)	- silty sand lens at ~24-1/2', loose, with charcoal inclusions and staining, fine to medium sand, grades to sandy clayey silt at ~26'	137	116	18				u 1.6
-8	26				129	108	20	48			
											t 0.7
-10	28			Silty SAND with gravel (SM) with sandy clay and sandy silt lenses: medium dense, brown (7.5YR4/4), wet, medium sand, some coarse, fine to coarse gravel ~1" diameter							
-12	30		(24)	- sandy lean clay lens at ~29-1/2'	134	110	21				
-14	32										
-16	34		31								
							21				
-18	36										
-20	38										
			4	- more gravels in drilling at ~39'							

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 101.0 ft
DEPTH TO WATER: 8.5 ft
BACKFILLED WITH: Grout
DRILLING DATE: April 23, 2010
GW measured in hollow-stem auger beofore beginning mud rotary

DRILLING METHOD: 6-inch-dia. Mud Rotary Wash
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Co.
LOGGED BY: K Robinson
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-1

Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE A-6a

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: See Plate 2 - Subsurface Exploration Location PlanN 1,979,586 E 6,058,062 SURFACE EL: 18 ft +/- (rel. NAVD 88 datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						- lens of sandy silt to silt with sand at ~40-1/2', soft			17				
-24	42					- gravels encountered in drilling between ~40-44'							
-26	44				(15)	Lean CLAY (CL): stiff, brown (7.5YR4/3), wet, with fine gravel at ~44-1/2', with charcoal inclusions, becomes sandy at ~46'	131 130	109 108	20 21		27	11	u 0.7 t 0.5 p 2.8
-28	46												
-30	48												
-32	50												
-34	52					Silty SAND (SM): medium dense, brown (7.5YR4/3), wet, medium sand, with some fine gravel and trace coarse gravel at ~56'							
-36	54				(40)								
-38	56							108	18				
-40	58					- ~1-1/2' gravel layer at ~57' during drilling							
-42	60				(44)	- grades to clayey sand (SC), gray (7.5YR5/1), with black striping ~1" thick, some iron oxide staining	128	112	15				
-44	62					Sandy Lean CLAY (CL): very stiff, dark gray to gray (7.5Y4/1 to 5/1), wet, with few fine dusky yellow gravels							p 4.5+
-46	64				(43)								
-48	66					- turns to clayey fine sand at ~65', medium dense, yellow brown (10YR5/4) with gray and olive gray (5Y5/2) pockets and lenses, some medium sand, some fine gravel	139	121	15				u 2.9
-50	68												
-52	70				(39)	Fat CLAY (CH): very stiff, greenish gray (10Y6/1), wet, with abundant iron oxide inclusions and staining	127	102	25				u 3.6
-54	72												p 4.5+
-56	74				(61)	Clayey SAND (SC): dense, olive gray to light olive gray (5Y5/2 to 7/2), wet, with iron oxide staining and charcoal staining, very fine sand	129 133	108 112	19 19				
-58	76												
-60	78				79	Poorly graded SAND with silt (SP-SM) with occasional gravel lenses: very dense, light yellow brown (10YR6/4), wet, with pockets of dark yellow brown							

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 101.0 ft
DEPTH TO WATER: 8.5 ft
BACKFILLED WITH: Grout
DRILLING DATE: April 23, 2010
GW measured in hollow-stem auger beofore beginning mud rotary

DRILLING METHOD: 6-inch-dia. Mud Rotary Wash
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Co.
LOGGED BY: K Robinson
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-1

Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE A-6b

LOCATION: See Plate 2 - Subsurface Exploration Location PlanN 1,979,586 E 6,058,062

SURFACE EL: 18 ft +/- (rel. NAVD 88 datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
				X		(10YR4/6), fine to medium sand			22				
-64	82												
-66	84												
				X	72					6			
-68	86					- gravel lens at ~86', ~3" thick, well-graded gravel with clay, yellow brown (10YR5/8), fine gravel ~1/4" diameter, some 1/2" diameter							
-70	88												
-72	90			X	65	- gravel lens at ~90', ~4" thick, silty sand with fine gravel, subrounded, yellow brown (10YR5/8)							
-74	92					- ~1' gravel lens at ~92' during drilling							
-76	94												
-78	96					- with few fine gravels ~1/2" diameter at ~95'							
-80	98					- ~3' of gravels and cobbles at ~97' during drilling							
-82	100					SILT (ML): very stiff, yellow brown (10YR5/8), wet, turns to sandy silt at ~100' with silt lenses - turns to gravelly silty clay (CL-ML) at ~100-1/2', fine gravel							
-84	102												
-86	104												
-88	106												
-90	108												
-92	110												
-94	112												
-96	114												
-98	116												
-100	118												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 101.0 ft
DEPTH TO WATER: 8.5 ft
BACKFILLED WITH: Grout
DRILLING DATE: April 23, 2010
GW measured in hollow-stem auger beofore beginning mud rotary

DRILLING METHOD: 6-inch-dia. Mud Rotary Wash
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Co.
LOGGED BY: K Robinson
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-1

Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE A-6c

LOCATION: See Plate 2 - Subsurface Exploration Location PlanN 1,979,554 E 6,058,123

SURFACE EL: 16 ft +/- (rel. NAVD 88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
14	2		Silty SAND (SM): dark brown (7.5YR3/2), moist, with gravel, grades to clayey sand/sandy lean clay at ~2', fine grained, subrounded							
12	4		Sandy Lean CLAY (CL): soft, brown (7.5YR4/2), moist, fine sand, turns to silty clay, dark brown with charcoal inclusions near shoe, dark brown, with roots, wet							
10	6	(5)			90	28	68			t 0.5
8	8		Silty CLAY with sand (CL-ML): stiff, brown (7.5YR4/2), wet, fine sand							
6	10	(16)		127	104	22		26	6	t 0.5
4	12									
2	14		Sandy Fat CLAY (CH): very stiff, brown (7.5YR4/4), wet, with gray brown mottles, with some fine gravel, charcoal inclusions, fine sand							
0	16	(25)			111	17				p 3.0
-2	18									
-4	20	(20)	Sandy Lean CLAY (CL): stiff, strong brown (7.5YR4/6), wet							
-6	22			99	85	17				
-8	24		- poorly-graded sand with clay between 20-25'							p 2.0
-10	26	(15)	Sandy SILT (ML): stiff, strong brown (7.5YR4/6), wet, with poorly-graded sand lens at 26.25', fine sand, with charcoal staining, strong brown lean clay at 26.5'							
-12	28									
-14	30									
-16	32									
-18	34									
-20	36									
-22	38									

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 26.5 ft
DEPTH TO WATER: 7.5 ft
BACKFILLED WITH: Cuttings
DRILLING DATE: April 22, 2010
GW measured ~24 hours after completion of drilling

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Co.
LOGGED BY: K Robinson
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-2

Sycamore Creek Enhancement Project
Santa Barbara, California

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: See Plate 2 - Subsurface Exploration Location PlanN 1,979,793 E 6,057,896 SURFACE EL: 21 ft +/- (rel. NAVD 88 datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
20						**ARTIFICIAL FILL (af)** Base Material: 4" asphalt concrete over approximately 7" base							
18	2					Silty SAND (SM)/Clayey SAND (SC): brown (7.5YR3/3), slightly moist, fine to medium sand, some fine grained							
16	4, 6				(10)	Silty SAND (SM): firm to loose, dark brown and brown (7.5YR4/4), moist, mottled, some black inclusions, fine sand	125	108	16	48			
14, 12, 10	8, 10				(3)	Sandy Lean CLAY (CL): very soft, brown (7.5YR4/3), wet, fine sand, with some gravel near 10'	123	97	27	70	21	1	t 0.6
8, 6	12, 14				(25)	Sandy, Silty CLAY (CL-ML): very stiff, brown (7.5YR4/2) to gray brown, wet, fine sand, fine to coarse gravel, some cobbles							
4	16					Clayey SAND with gravel (SC) to Well-graded GRAVEL with clay (GW-GC): medium dense, brown, wet			11				
2	18					Sandy, Silty CLAY (CL-ML): firm, brown (7.5YR4/2), wet, trace fine gravel, fine sand							
0, -2, -4	20, 22, 24				(11) (34)	Silty SAND (SM): brown, trace fine charcoal and red inclusions							
-6, -8, -10, -12, -14, -16, -18	26, 28, 30, 32, 34, 36, 38												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 25.0 ft
DEPTH TO WATER: 5.3 ft
BACKFILLED WITH: Cuttings, patched with concrete dyed black
DRILLING DATE: April 23, 2010
GW measured in adjacent CPT

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Co.
LOGGED BY: K Robinson
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-3

Sycamore Creek Enhancement Project
Santa Barbara, California

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLES	BLOW COUNT / REC"/DRIVE"	LOCATION: The drill hole location referencing local landmarks or coordinates SURFACE EL: Using local, MSL, MLLW or other datum MATERIAL DESCRIPTION	
-12	2		1		25	Well graded GRAVEL (GW)	COARSE GRAINED
-14	4		2		(25)	Poorly graded GRAVEL (GP)	COARSE GRAINED
-16	6					Well graded SAND (SW)	COARSE GRAINED
-18	8		3		(25)	Poorly graded SAND (SP)	COARSE GRAINED
-20	10		4		(25)	Silty SAND (SM)	COARSE GRAINED
-22	12		5		18"/ 30"	Clayey SAND (SC)	COARSE GRAINED
-24	14		6			Silty, Clayey SAND (SC-SM)	COARSE GRAINED
-26	16					Elastic SILT (MH)	FINE GRAINED
-28	18		7			SILT (ML)	FINE GRAINED
-30	20		8		20"/ 24"	Silty CLAY (CL-ML)	FINE GRAINED
-32	22		9		(25)	Fat CLAY (CH)	FINE GRAINED
-34	24		10		30"/ 30"	Lean CLAY (CL)	FINE GRAINED
-36	26					CONGLOMERATE	ROCK
-38	28		11		20"/ 24"	SANDSTONE	ROCK
-40	30		12			SILTSTONE	ROCK
-42	32		13			MUDSTONE	ROCK
-44	34					CLAYSTONE	ROCK
-46	36					BASALT	ROCK
-48	38					ANDESITE BRECCIA	ROCK
						Paving and/or Base Materials	

General Notes

Soil Texture Symbol

Sloped line in symbol column indicates transitional boundary

Samplers and sampler dimensions
(unless otherwise noted in report text) are as follows:

Symbol for:

1	SPT Sampler, driven 1-3/8" ID, 2" OD
2	CA Liner Sampler, driven 2-3/8" ID, 3" OD
3	CA Liner Sampler, disturbed 2-3/8" ID, 3" OD
4	Thin-walled Tube, pushed 2-7/8" ID, 3" OD
5	Bulk Bag Sample (from cuttings)
6	CA Liner Sampler, Bagged
7	Hand Auger Sample
8	CME Core Sample
9	Pitcher Sample
10	Lexan Sample
11	Vibracore Sample
12	No Sample Recovered
13	Sonic Soil Core Sample

Sampler Driving Resistance

Number of blows with 140 lb. hammer, falling 30" to drive sampler 1 ft. after seating sampler 6"; for example,

Blows/ft	Description
25	25 blows drove sampler 12" after initial 6" of seating
86/11"	After driving sampler the initial 6" of seating, 36 blows drove sampler through the second 6" interval, and 50 blows drove the sampler 5" into the third interval
50/6"	50 blows drove sampler 6" after initial 6" of seating
Ref/3"	50 blows drove sampler 3" during initial 6" seating interval

Blow counts for California Liner Sampler shown in ( )

Length of sample symbol approximates recovery length

Classification of Soils per ASTM D2487 or D2488

Geologic Formation noted in bold font at the top of interpreted interval

Strength Legend

Q = Unconfined Compression
u = Unconsolidated Undrained Triaxial
t = Torvane
p = Pocket Penetrometer
m = Miniature Vane

Water Level Symbols

- ▽ Initial or perched water level
- ▼ Final ground water level
- ∿ Seepages encountered

Rock Quality Designation (RQD) is the sum of recovered core pieces greater than 4 inches divided by the length of the cored interval.

KEY TO TERMS & SYMBOLS USED ON LOGS

PLATE A-9

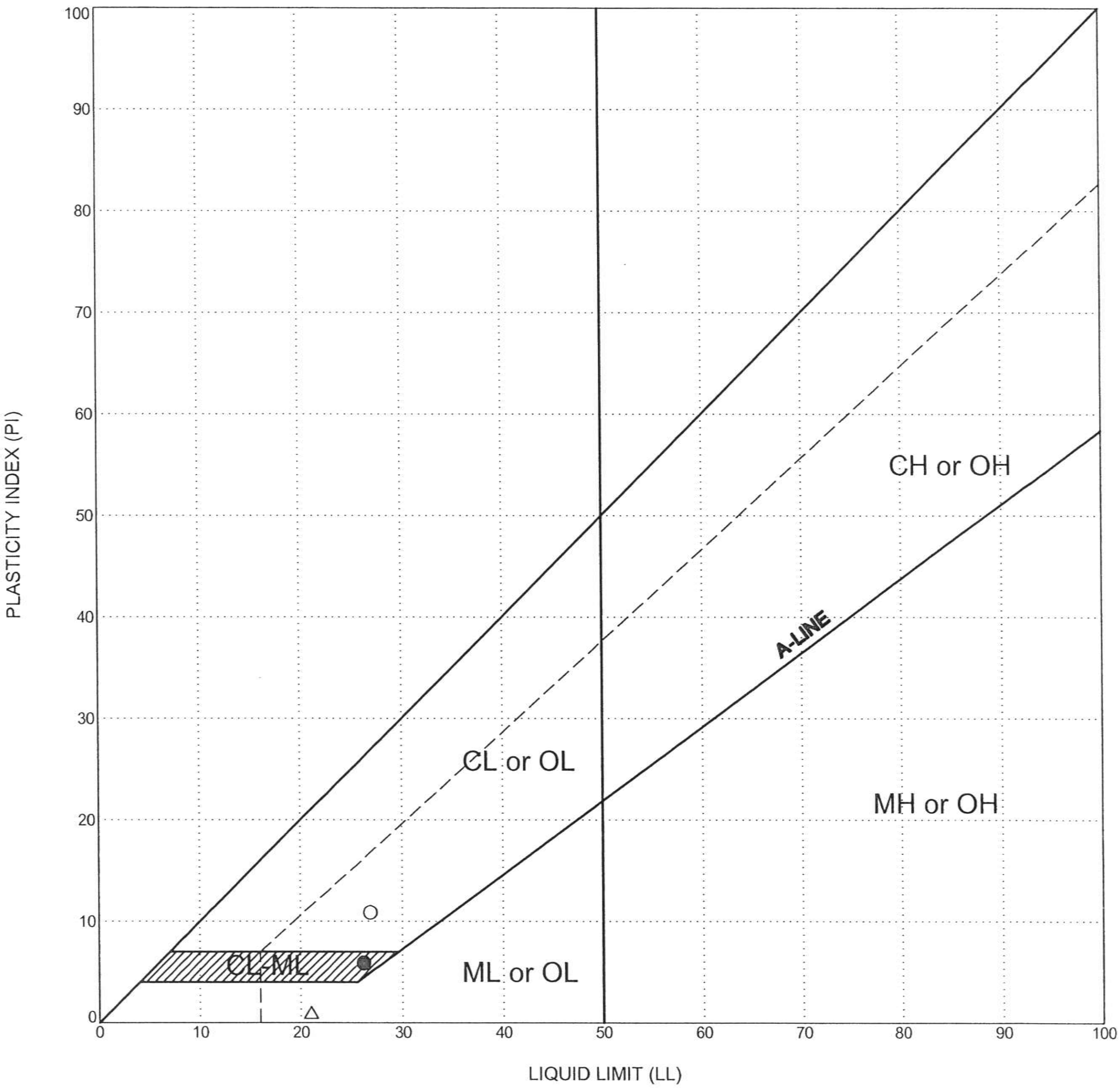
APPENDIX B
LABORATORY TESTING

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC %	FINES %	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	COMPACTION TEST MAX DD pcf	COMPACTION TEST OPT MC %	DIRECT SHEAR C ksf	DIRECT SHEAR PHI deg	COMPRESSIVE STRENGTH TESTS Qu, ksf	COMPRESSIVE STRENGTH TESTS S_u (Cell Prs.) ksf	CORROSIVITY TESTS R	CORROSIVITY TESTS pH	CORROSIVITY TESTS Cl	CORROSIVITY TESTS So_4 (%)	R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	SPECIFIC GRAVITY
DH-1	5.0	1A	Sandy Lean CLAY (CL)	129	112	15																	
DH-1	6.0	1B	Lean CLAY (CL) to Silty CLAY (CL-ML)																				
DH-1	9.0																						
DH-1	10.0		Sandy Lean CLAY (CL)	126	101	25	61																
DH-1	10.5		Silty SAND (SM)		107	19						0.5	31										
DH-1	11.0																						
DH-1	15.0	3A	Clayey SAND (SC)				22																
DH-1	15.5	3B	Clayey SAND (SC)	133	114	17																	
DH-1	25.0	4A	Silty SAND (SM)	137	116	18									1.6(3)								
DH-1	25.5	4B	Silty SAND (SM)	129	108	20																	
DH-1	26.0	4C	Silty SAND (SM)				48																
DH-1	27.0																						
DH-1	30.0	5A	Sandy Lean CLAY (CL)	134	110	21																	
DH-1	30.5	5B	Silty SAND (SM)																				
DH-1	35.0	6A	Silty SAND with gravel (SM)			21																	
DH-1	40.0	7A	Silty SAND with gravel (SM)			17																	
DH-1	45.0	8A	Lean CLAY (CL)	131	109	20									0.7(5.4)								
DH-1	45.5	8B	Lean CLAY (CL)	130	108	21		27	11														
DH-1	46.0																						
DH-1	55.5		Silty SAND (SM)		108	18						0.2	38										
DH-1	60.0	10A	Clayey SAND (SC)	128	112	15																	
DH-1	61.0																						
DH-1	65.0	11A	Sandy Lean CLAY (CL)	139	121	15									2.9(7.8)								
DH-1	70.0	12A	Fat CLAY (CH)	127	102	25									3.6(7.8)								
DH-1	71.0																						
DH-1	75.0	13A	Clayey SAND (SC)	129	108	19																	
DH-1	75.5	13B	Clayey SAND (SC)	133	112	19																	
DH-1	80.0	14	Poorly-graded SAND with silt (SP-SM)			22																	
DH-1	85.0	15A	Poorly-graded SAND with silt (SP-SM)				6																
DH-2	5.0																						

SUMMARY OF LABORATORY TEST RESULTS
Sycamore Creek Enhancement Project
Santa Barbara, California

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC %	FINES %	ATTERBERG LIMITS		COMPACTION TEST		DIRECT SHEAR		COMPRESSIVE STRENGTH TESTS		CORROSIVITY TESTS				R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	SPECIFIC GRAVITY
								LL	PI	MAX DD pcf	OPT MC %	C ksf	PHI deg	Qu, ksf	S_u (Cell Prs.) ksf	R	pH	Cl	So_4 (%)				
DH-2	5.5	1	Clayey SAND (SC)		90	28	68					0.1	34										
DH-2	6.0																						
DH-2	10.5	2	Silty CLAY with sand (CL-ML)	127	104	22		26	6														
DH-2	11.0																						
DH-2	15.5	3A	Sandy Fat CLAY (CH)		111	17						0.5	23										
DH-2	16.0																						
DH-2	21.0	4B	Sandy Lean CLAY (CL)	99	85	17																	
DH-2	24.0																						
DH-3	5.0	1	Silty SAND (SM)	125	108	16	48																
DH-3	9.0																						
DH-3	9.5	2A	Sandy SILT (ML)	123	97	27	70	21	1			0.0	37										
DH-3	10.0	2B	Sandy Lean CLAY (CL)									0.2	31										
DH-3	15.0	3B	Clayey SAND with gravel (SC)			11																	

SUMMARY OF LABORATORY TEST RESULTS
Sycamore Creek Enhancement Project
Santa Barbara, California

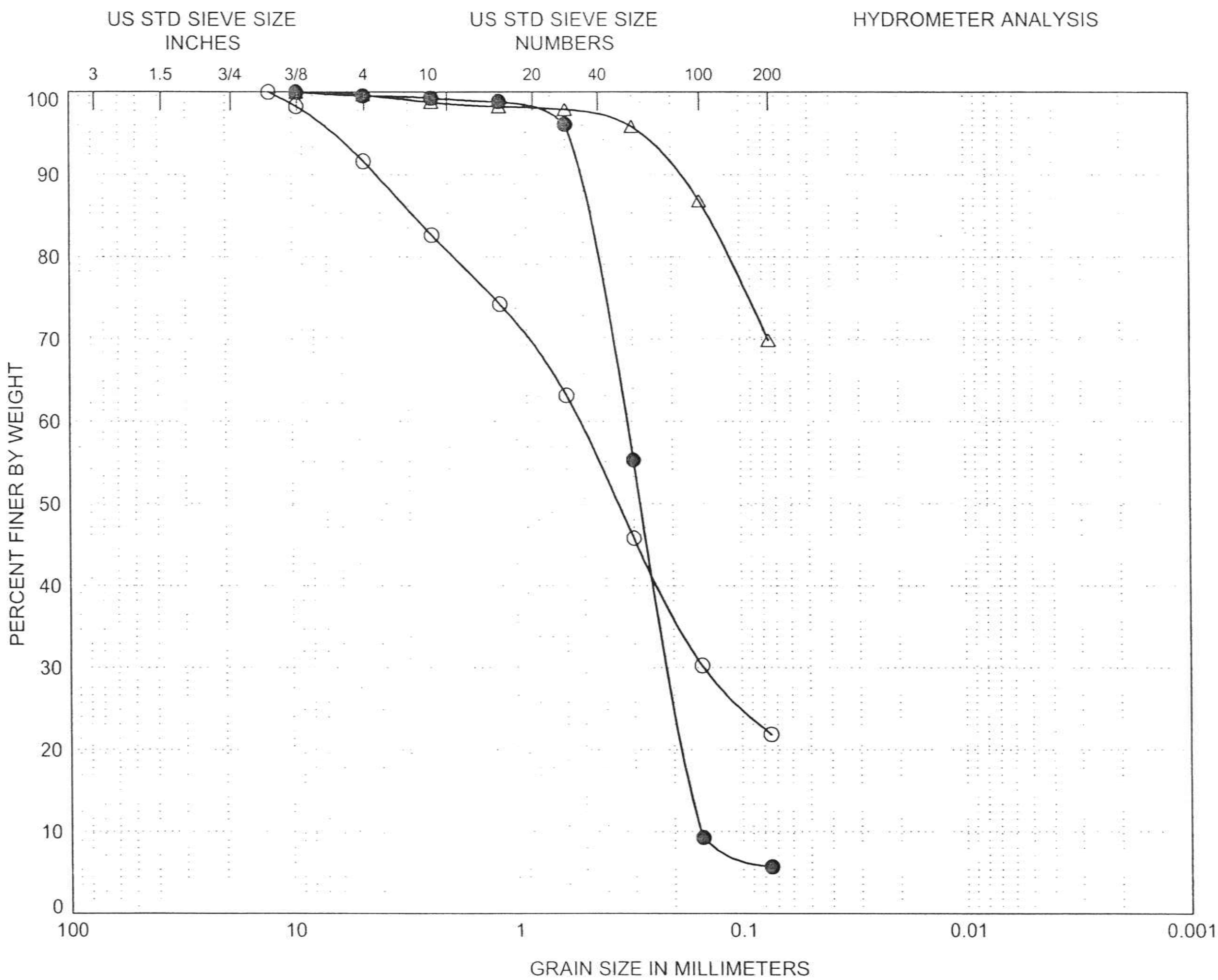


LEGEND location	depth, ft	CLASSIFICATION	LIQUID LIMIT(LL)	PLASTIC LIMIT(PL)	PLASTICITY INDEX (PI)
○ DH-1	45.5	Lean CLAY (CL)	27	16	11
● DH-2	10.5	Silty CLAY with sand (CL-ML)	26	20	6
△ DH-3	9.5	Sandy SILT (ML)	21	20	1

(ATTERBERG LIMITS TEST RESULTS)

PLASTICITY CHART
Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE B-2

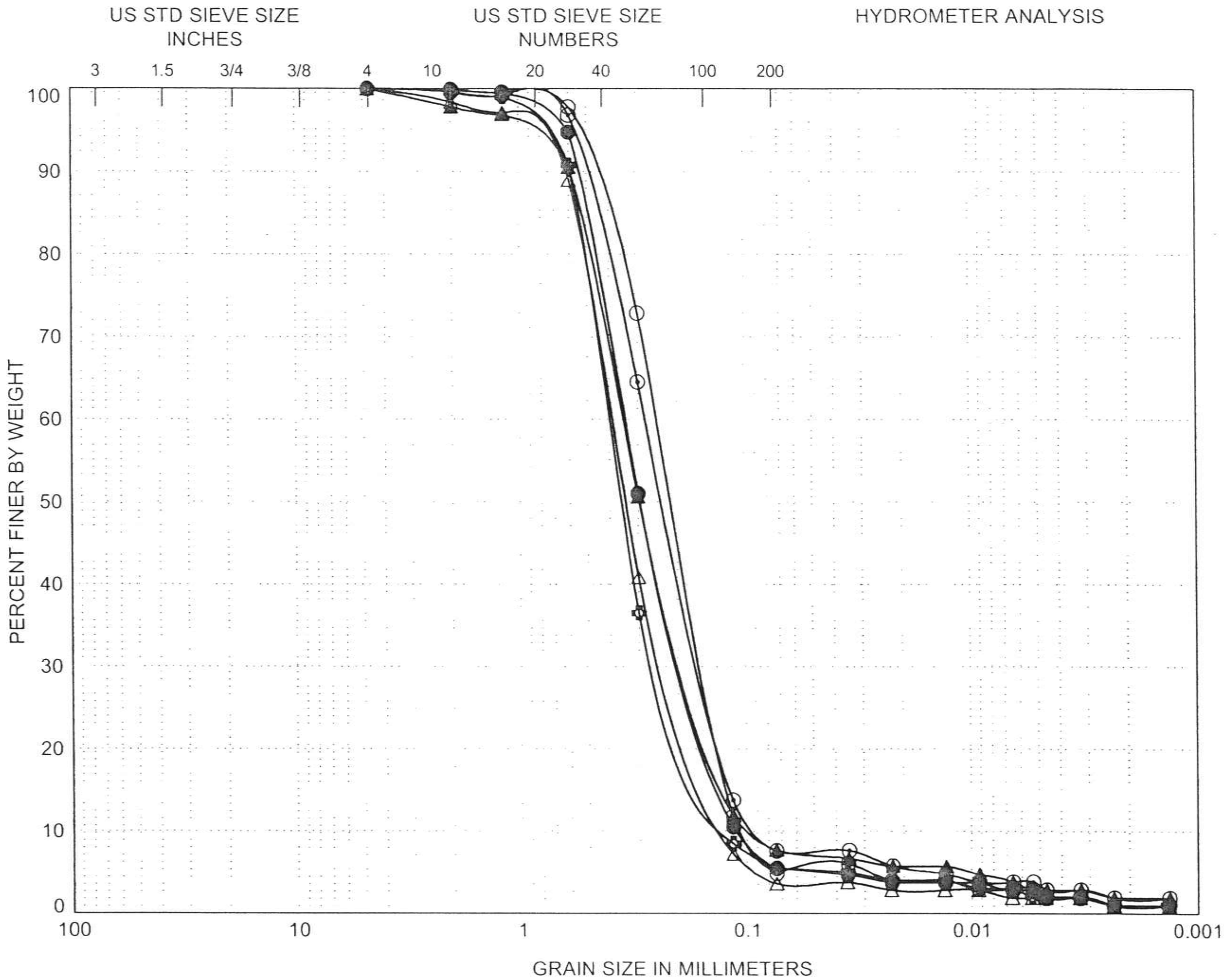


GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

	LEGEND (location)	(depth,ft)	CLASSIFICATION	Cc	Cu
○	DH-1	15.0	Clayey SAND (SC)		
●	DH-1	85.0	Poorly-graded SAND with silt (SP-SM)	0.9	2.1
△	DH-3	9.5	Sandy SILT (ML)		

GRAIN SIZE CURVES
Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE B-3a

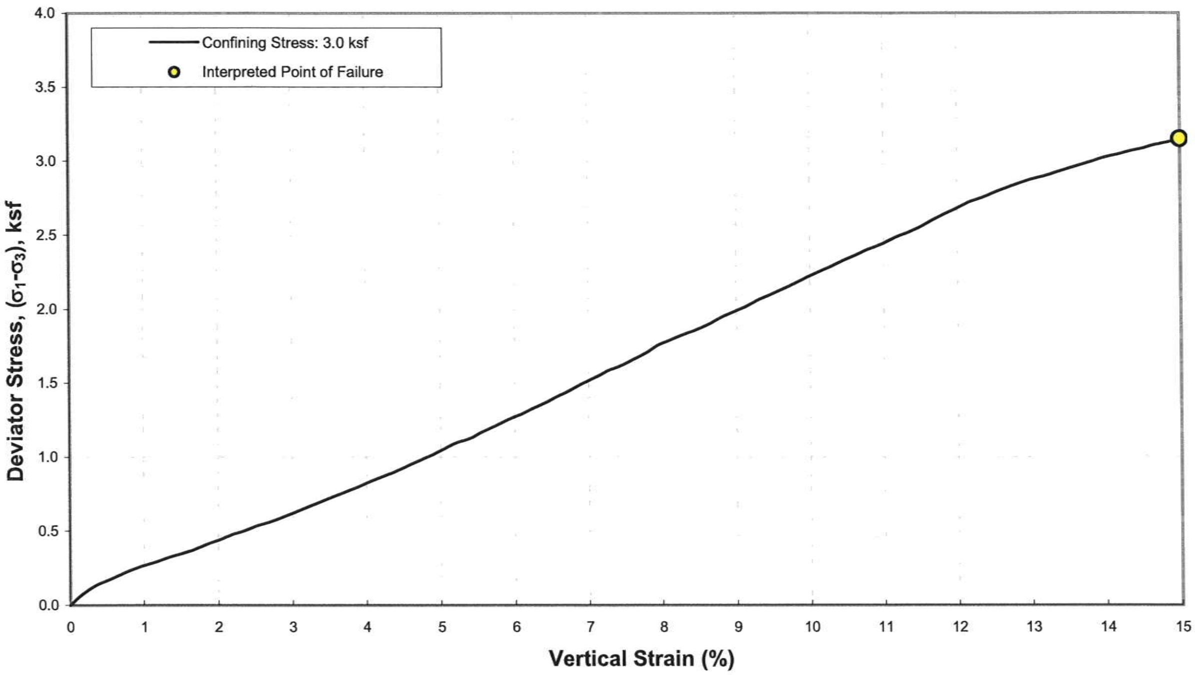


GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

LEGEND	(location)	(depth,ft)	CLASSIFICATION	Cc	Cu
○	G-1	0.0	Poorly graded SAND (SP)	1.2	3.1
●	G-2	0.0	Poorly graded SAND with silt (SP-SM)	0.9	3.1
△	G-3	0.0	Poorly graded SAND (SP)	1.0	3.2
▲	G-4	0.0	Poorly graded SAND with silt (SP-SM)	1.0	3.7
⊙	G-5	0.0	Poorly graded SAND with silt (SP-SM)	1.6	5.0
✪	G-6	0.0	Poorly graded SAND with silt (SP-SM)	1.2	3.3

GRAIN SIZE CURVES
Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE B-3b



SAMPLE ID

Boring Number:	DH-01
Sample Number:	#4A
Sample Depth:	25.0ft
USCS Classification:	Silty SAND (SM): brown, wet
Sample Type:	Ring

SAMPLE PROPERTIES

Water Content, %	18.2
Wet Density, pcf	137.1
Dry Density, pcf	116.0
Saturation, %	100
Void Ratio	0.51
Diameter, in	2.367
Height, in	4.950
Height/Diameter	2.1

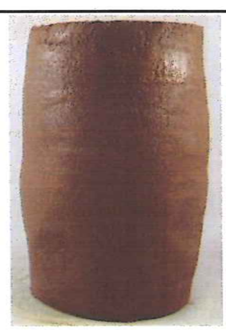
CLASSIFICATION

Sieve Size	% Passing	Other Parameters	
3/8-in. (9.5mm)	---	Liquid Limit	---
No. 4 (4.75mm)	---	Plastic Limit	---
No. 10 (2.0mm)	---	Plasticity Index	---
No. 30 (0.6mm)	---	Estimated Gs	2.81
No. 100 (0.150mm)	---		
No. 200 (0.075mm)	---		

TEST SUMMARY

Strain Rate, %/min	1.01
Cell Pressure, ksf	3.0
Deviator Stress at Failure, ksf	3.2
Undrained Shear Strength, ksf	1.6
Axial Strain at Failure, %	15.0
Tested By:	JC
Date Tested:	05.17.10

SAMPLE IMAGES



REMARKS

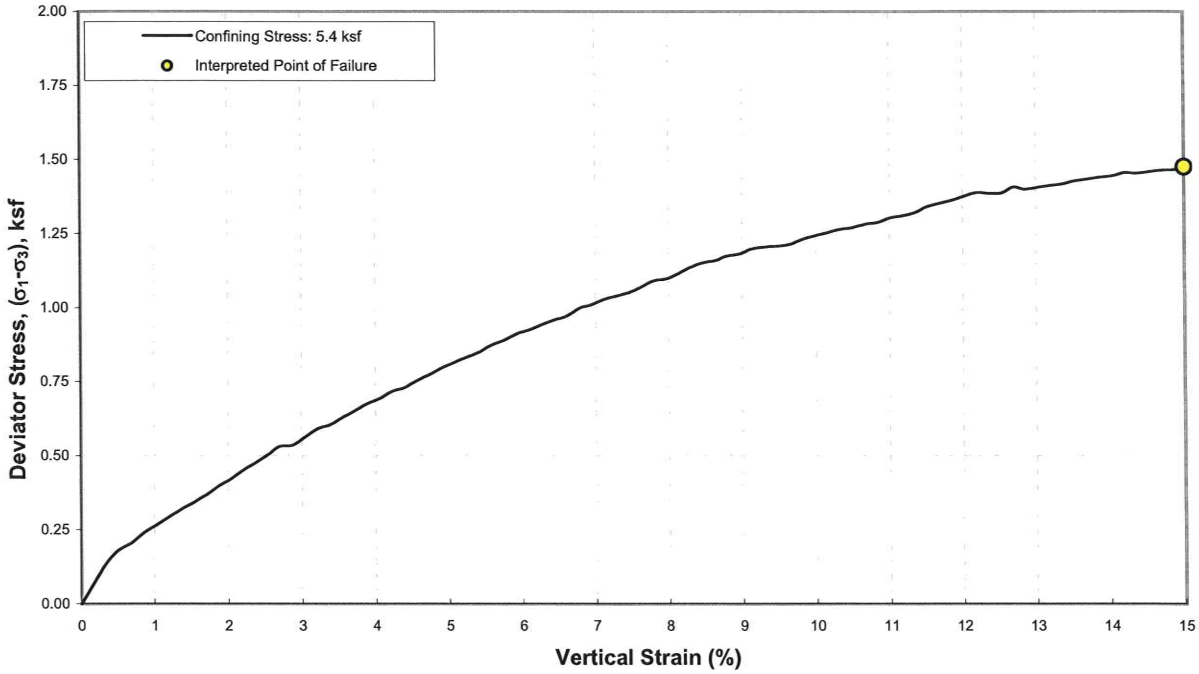
Test Method: ASTM 2850.

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE B-4a

Penfield & Smith
Project No. 3037.047



SAMPLE ID

Boring Number:	DH-01
Sample Number:	#8A
Sample Depth:	45.0ft
USCS Classification:	Lean CLAY (CL): dark brown, wet
Sample Type:	Ring

CLASSIFICATION

Sieve Size	% Passing
3/8-in. (9.5mm)	---
No. 4 (4.75mm)	---
No. 10 (2.0mm)	---
No. 30 (0.6mm)	---
No. 100 (0.150mm)	---
No. 200 (0.075mm)	---

Other Parameters	
Liquid Limit	---
Plastic Limit	---
Plasticity Index	---
Estimated Gs	2.7

SAMPLE PROPERTIES

Water Content, %	20.1
Wet Density, pcf	131.4
Dry Density, pcf	109.5
Saturation, %	100
Void Ratio	0.54
Diameter, in	2.388
Height, in	5.020
Height/Diameter	2.1

TEST SUMMARY

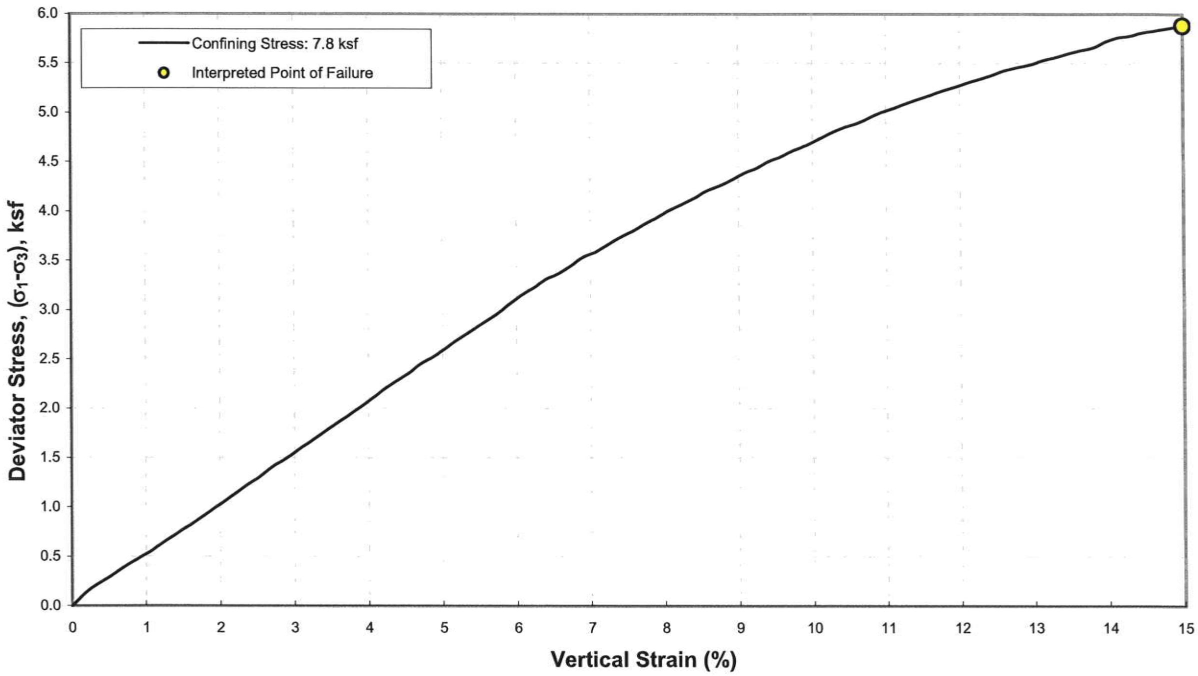
Strain Rate, %/min	1.00
Cell Pressure, ksf	5.4
Deviator Stress at Failure, ksf	1.5
Undrained Shear Strength, ksf	0.7
Axial Strain at Failure, %	15.0
Tested By:	JC
Date Tested:	05.18.10

SAMPLE IMAGES

REMARKS

Test Method: ASTM 2850.

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST
Sycamore Creek Enhancement Project
Santa Barbara, California



SAMPLE ID

Boring Number:	DH-01
Sample Number:	#11A
Sample Depth:	65.0ft
USCS Classification:	Sandy Lean CLAY (CL): dark yellowish brown, wet
Sample Type:	Ring

SAMPLE PROPERTIES

Water Content, %	14.8
Wet Density, pcf	138.5
Dry Density, pcf	120.6
Saturation, %	100
Void Ratio	0.40
Diameter, in	2.404
Height, in	4.940
Height/Diameter	2.1

CLASSIFICATION

Sieve Size	% Passing	Other Parameters	
3/8-in. (9.5mm)	---	Liquid Limit	---
No. 4 (4.75mm)	---	Plastic Limit	---
No. 10 (2.0mm)	---	Plasticity Index	---
No. 30 (0.6mm)	---	Estimated Gs	2.71
No. 100 (0.150mm)	---	S_u from T_v, ksf	---
No. 200 (0.075mm)	---	S_u from PP, ksf	2.8

TEST SUMMARY

Strain Rate, %/min	1.00
Cell Pressure, ksf	7.8
Deviator Stress at Failure, ksf	5.9
Undrained Shear Strength, ksf	2.9
Axial Strain at Failure, %	15.0
Tested By:	JC
Date Tested:	05.20.10

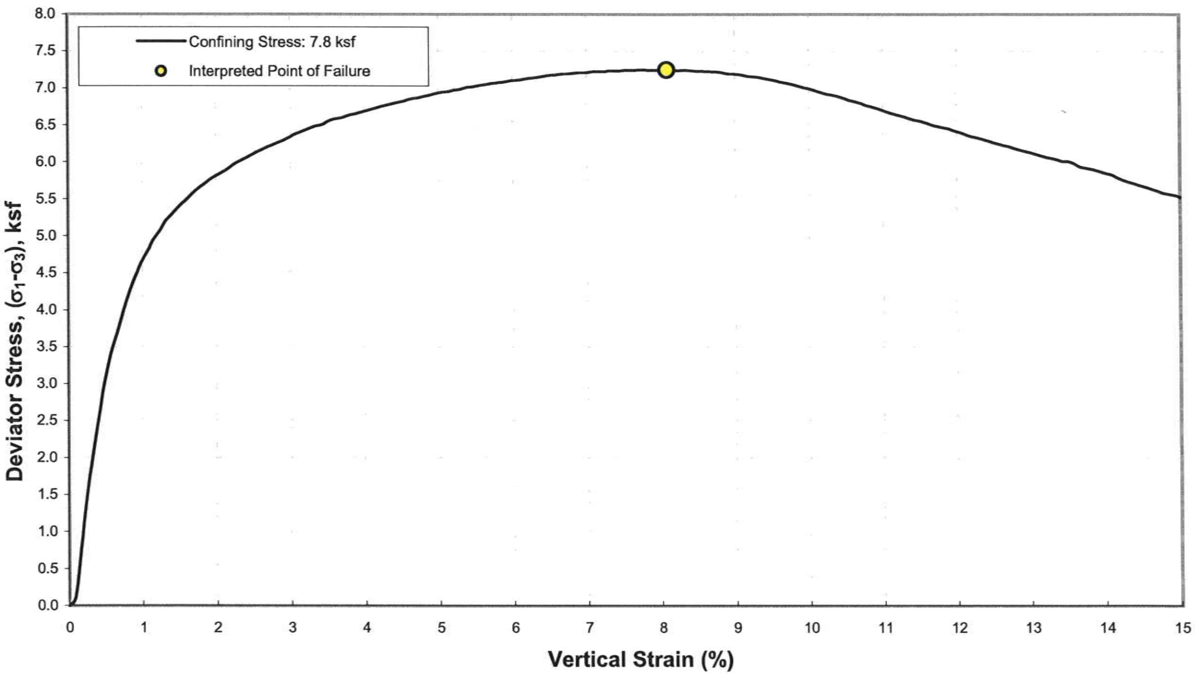
SAMPLE IMAGES

REMARKS

Test Method: ASTM 2850.

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

Sycamore Creek Enhancement Project
Santa Barbara, California



SAMPLE ID

Boring Number:	DH-01
Sample Number:	#12A
Sample Depth:	70.0ft
USCS Classification:	Fat CLAY (CH): olive gray with FeO2 mottling, wet
Sample Type:	Ring

SAMPLE PROPERTIES

Water Content, %	24.7
Wet Density, pcf	127.2
Dry Density, pcf	102.0
Saturation, %	100
Void Ratio	0.68
Diameter, in	2.396
Height, in	5.030
Height/Diameter	2.1

CLASSIFICATION

Sieve Size	% Passing
3/8-in. (9.5mm)	---
No. 4 (4.75mm)	---
No. 10 (2.0mm)	---
No. 30 (0.6mm)	---
No. 100 (0.150mm)	---
No. 200 (0.075mm)	---

Other Parameters	
Liquid Limit	---
Plastic Limit	---
Plasticity Index	---
Estimated Gs	2.74
S_u from T_v, ksf	---
S_u from PP, ksf	4.5+

TEST SUMMARY

Strain Rate, %/min	0.51
Cell Pressure, ksf	7.8
Deviator Stress at Failure, ksf	7.2
Undrained Shear Strength, ksf	3.6
Axial Strain at Failure, %	8.1
Tested By:	JC
Date Tested:	05.19.10

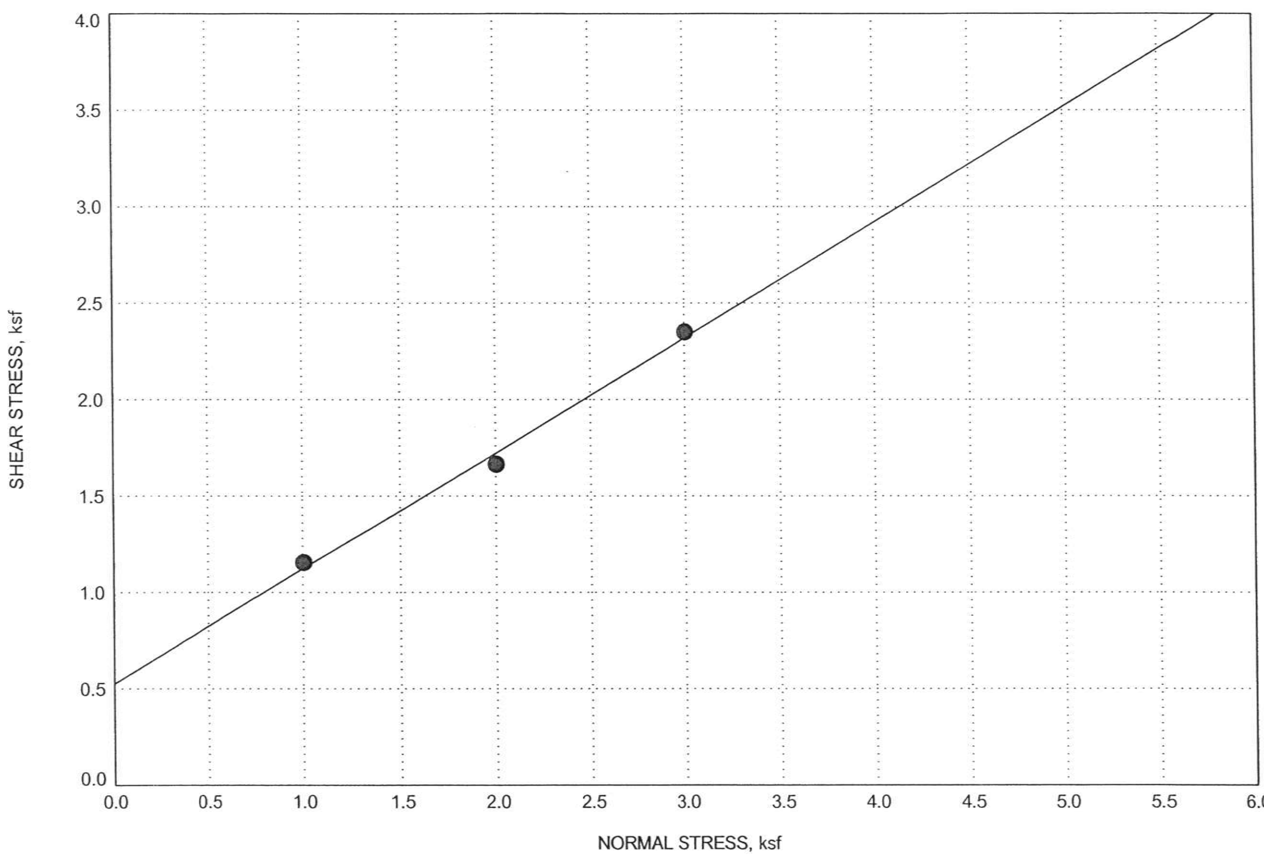
SAMPLE IMAGES

REMARKS

Test Method: ASTM 2850.
Effective stress lower than requested: test run at overburden for 65.0ft.

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

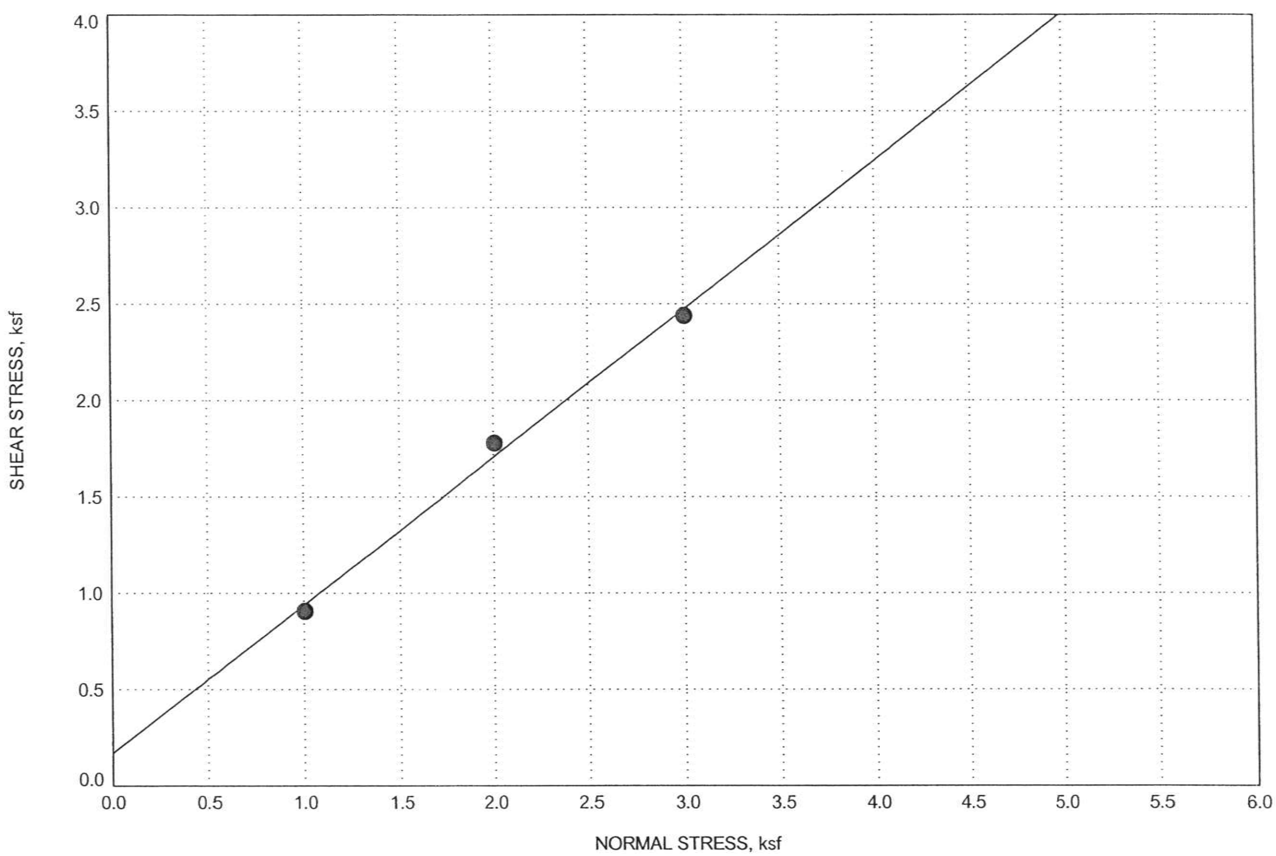
Sycamore Creek Enhancement Project
Santa Barbara, Callifornia



COHESION, ksf	0.5
ANGLE OF INTERNAL FRICTION, deg	31
LOCATION	DH-1
DEPTH, ft	10.5
MOISTURE CONTENT, %	19
UNIT DRY WEIGHT, pcf	107
MATERIAL DESCRIPTION	Silty SAND (SM)
SAMPLE CONDITION	Driven Ring

DIRECT SHEAR TEST RESULTS
Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE B-5a

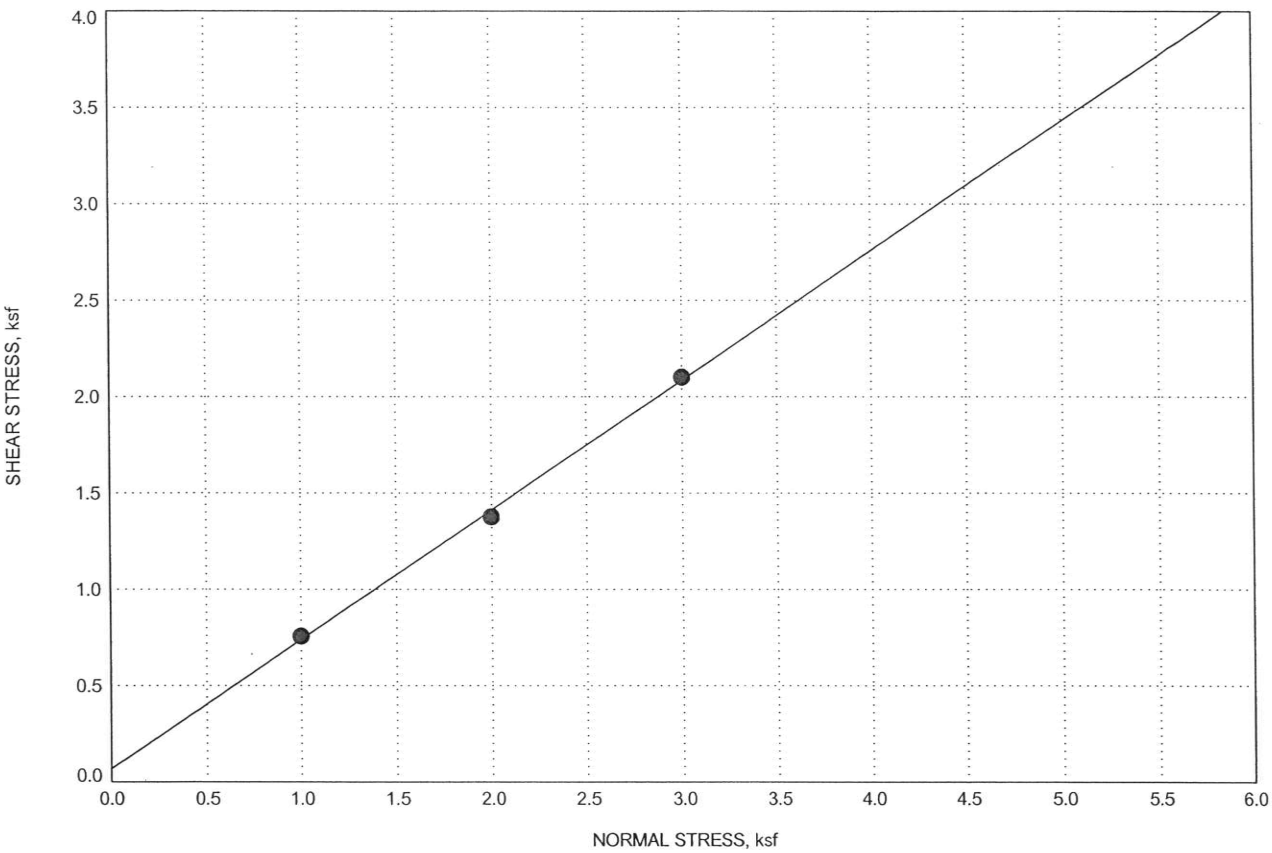


COHESION, ksf	0.2
ANGLE OF INTERNAL FRICTION, deg	38

LOCATION	DH-1
DEPTH, ft	55.5
MOISTURE CONTENT, %	18
UNIT DRY WEIGHT, pcf	108
MATERIAL DESCRIPTION	Silty SAND (SM)
SAMPLE CONDITION	Driven Ring

DIRECT SHEAR TEST RESULTS
Sycamore Creek Enhancement Project
Santa Barbara, California

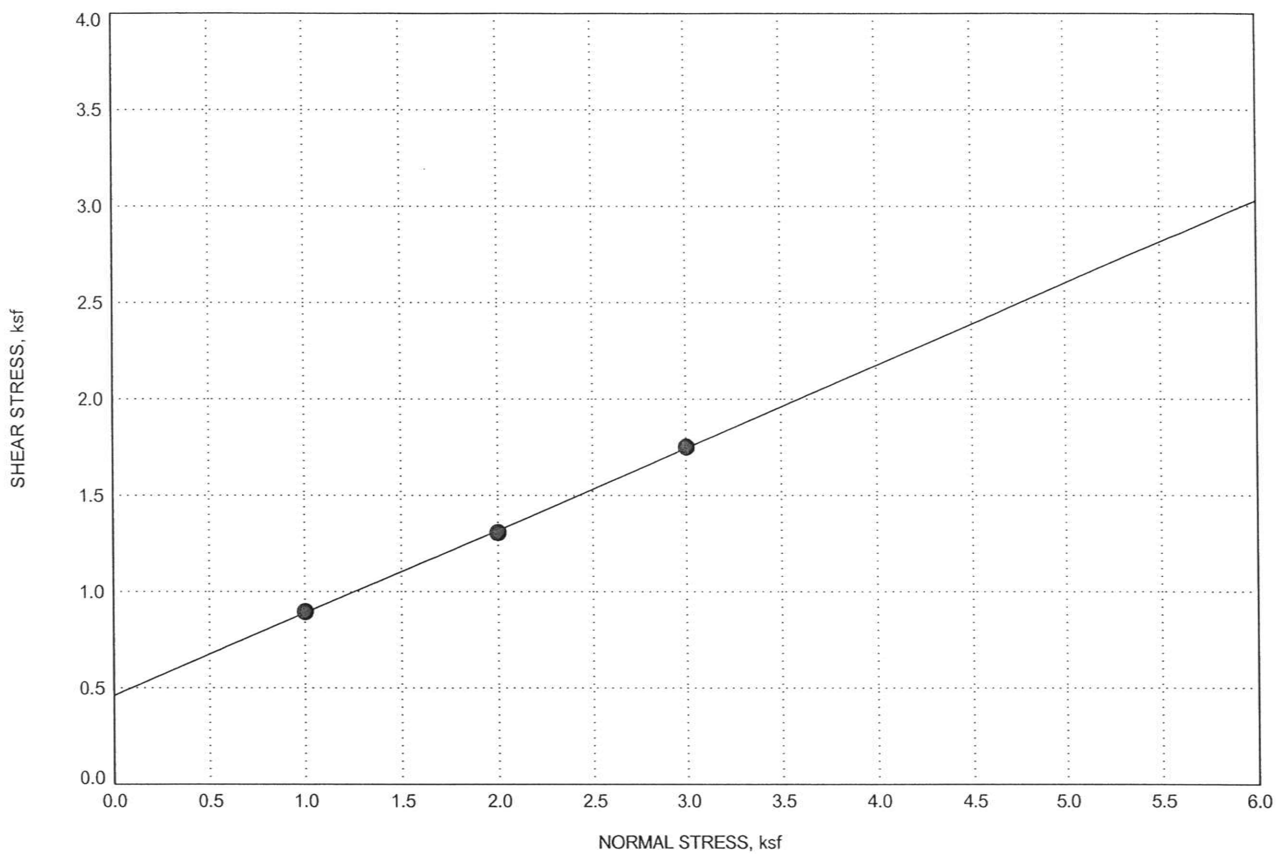
PLATE B-5b



COHESION, ksf	0.1
ANGLE OF INTERNAL FRICTION, deg	34
LOCATION	DH-2
DEPTH, ft	5.5
MOISTURE CONTENT, %	28
UNIT DRY WEIGHT, pcf	90
MATERIAL DESCRIPTION	Clayey SAND (SC)
SAMPLE CONDITION	Driven Ring

DIRECT SHEAR TEST RESULTS
Sycamore Creek Enhancement Project
Santa Barbara, California

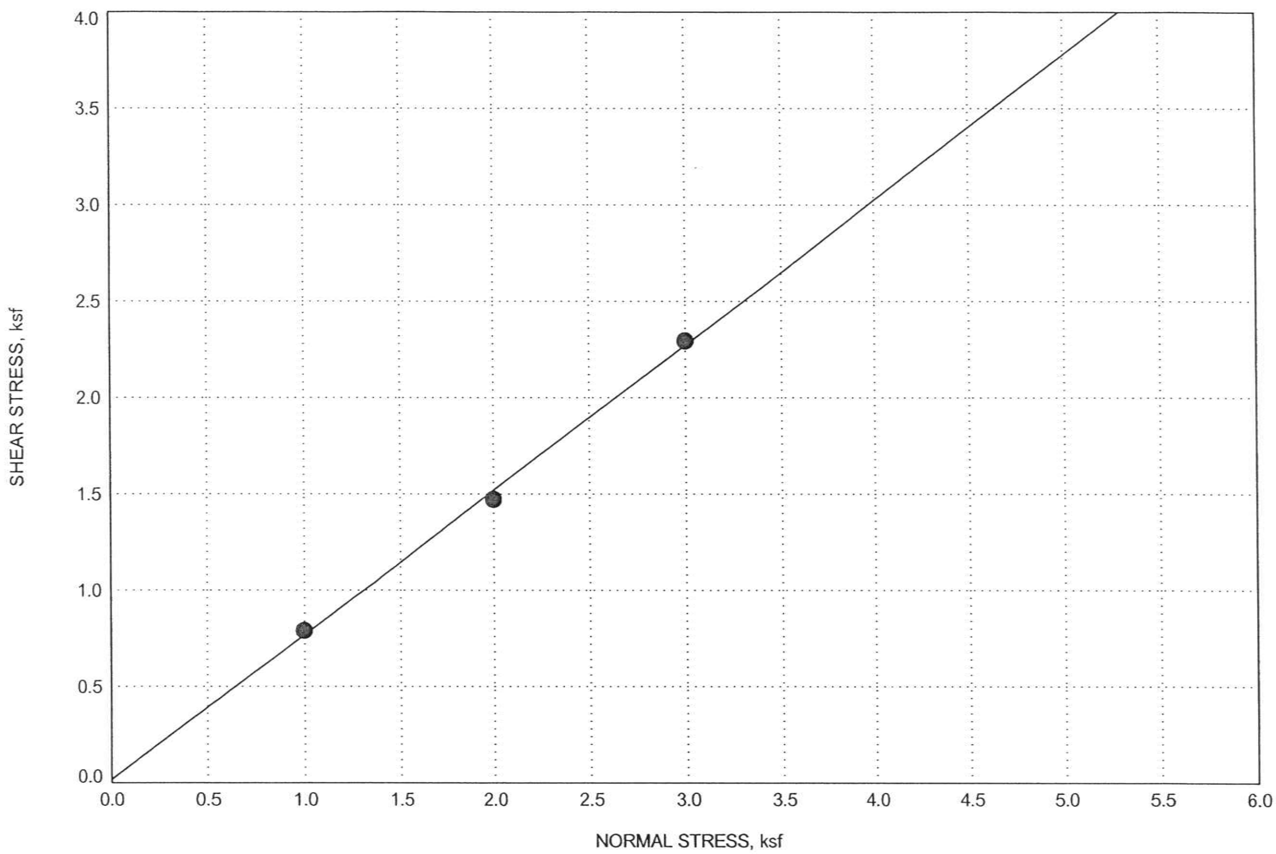
PLATE B-5c



COHESION, ksf	0.5
ANGLE OF INTERNAL FRICTION, deg	23
LOCATION	DH-2
DEPTH, ft	15.5
MOISTURE CONTENT, %	17
UNIT DRY WEIGHT, pcf	111
MATERIAL DESCRIPTION	Sandy Fat CLAY (CH)
SAMPLE CONDITION	Driven Ring

DIRECT SHEAR TEST RESULTS
Sycamore Creek Enhancement Project
Santa Barbara, California

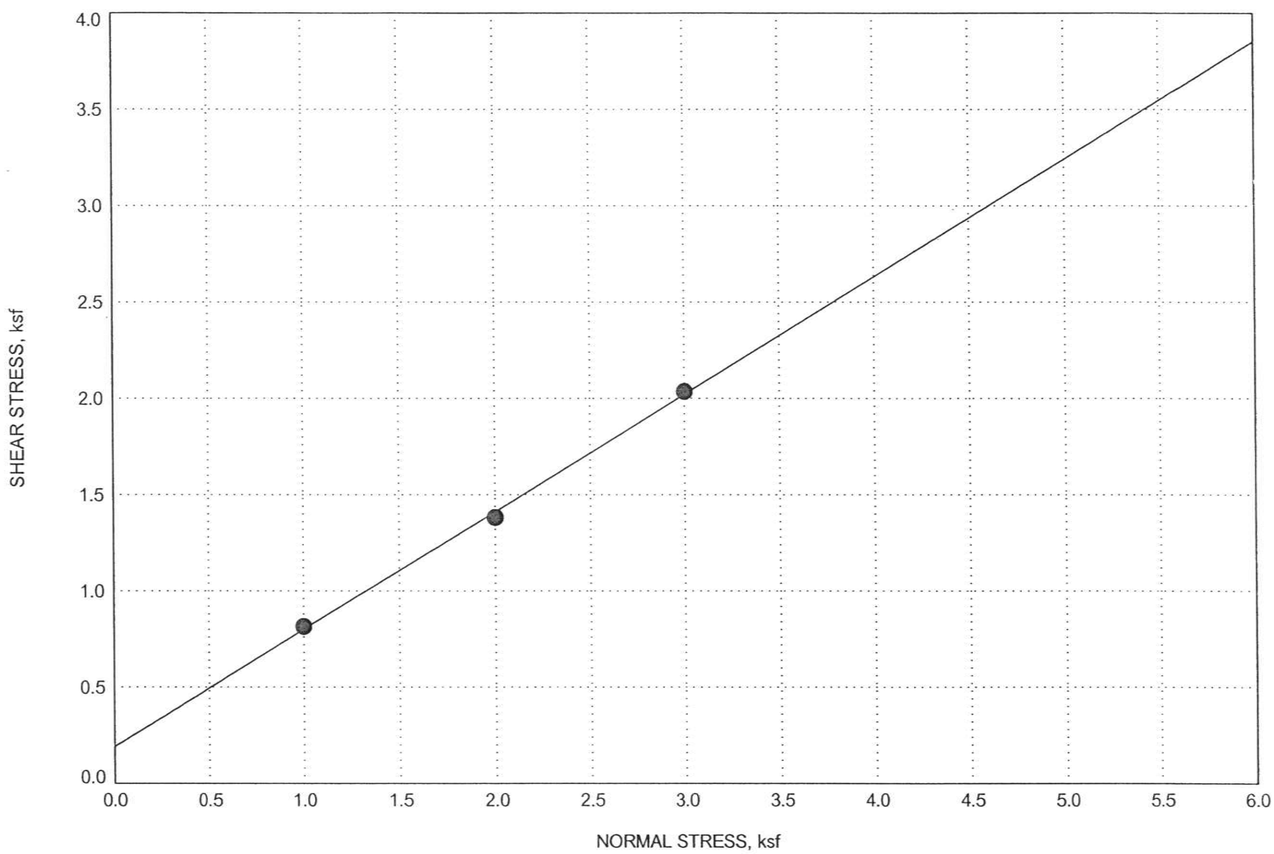
PLATE B-5d



COHESION, ksf	0.0
ANGLE OF INTERNAL FRICTION, deg	37
LOCATION	DH-3
DEPTH, ft	9.5
MOISTURE CONTENT, %	27
UNIT DRY WEIGHT, pcf	97
MATERIAL DESCRIPTION	Sandy SILT (ML)
SAMPLE CONDITION	Ring Sample

DIRECT SHEAR TEST RESULTS
Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE B-5e



COHESION, ksf	0.2
ANGLE OF INTERNAL FRICTION, deg	31
LOCATION	DH-3
DEPTH, ft	10
MOISTURE CONTENT, %	
UNIT DRY WEIGHT, pcf	
MATERIAL DESCRIPTION	Sandy Lean CLAY (CL)
SAMPLE CONDITION	Driven Ring

DIRECT SHEAR TEST RESULTS
Sycamore Creek Enhancement Project
Santa Barbara, California

PLATE B-5f

COOPER

Corrosivity Test Summary

CTL #: 446-084
Client: Fugro West
Remarks:

Date: 5/14/2010
Project: Sycamore Creek

Tested By: PJ

Checked: PJ
Proj. No: 3037.047

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	As Received	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
DH-1	5B	30.5	-	3089	-	15	<5	<0.0005	8.3	-	15.7	Reddish Brown Silty SAND

APPENDIX C
CALTRANS (2007) LOTB's

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
05	SB	101	17.4/20.6	638	652

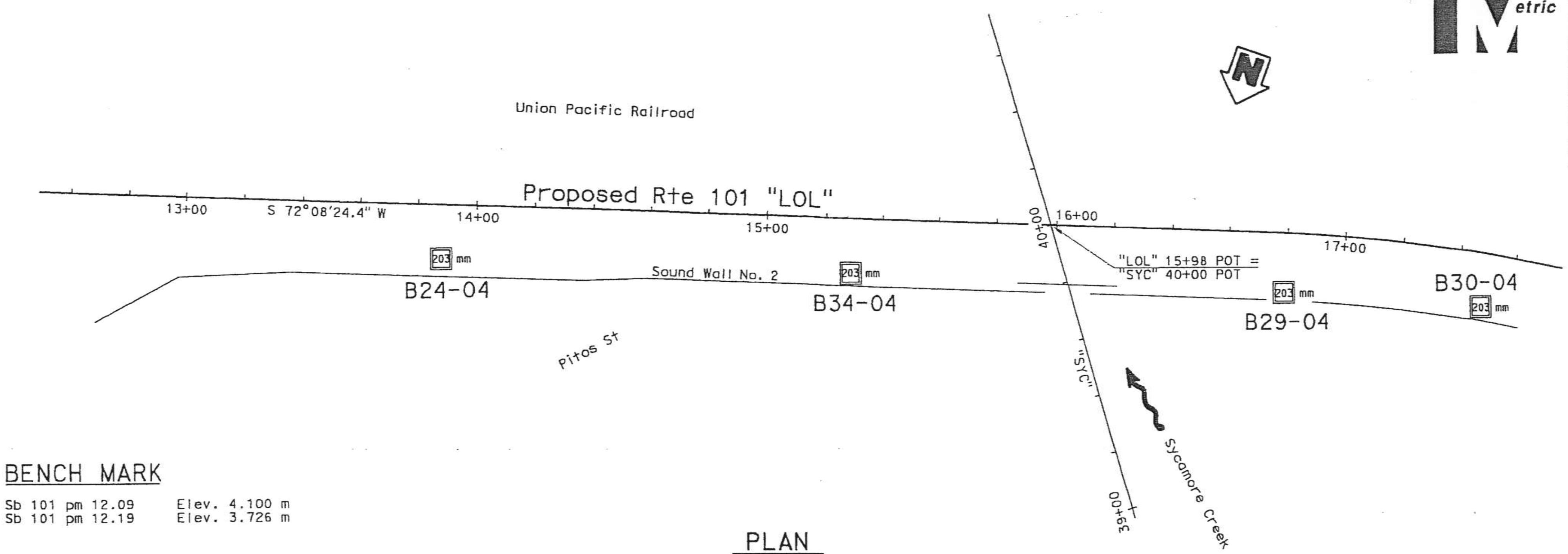
Caltrans Metric

REGISTERED CIVIL ENGINEER 6-22-05

9-17-07
PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

REGISTERED PROFESSIONAL ENGINEER Sara von Schwind No. C050789 Exp. 9-30-05 CIVIL STATE OF CALIFORNIA



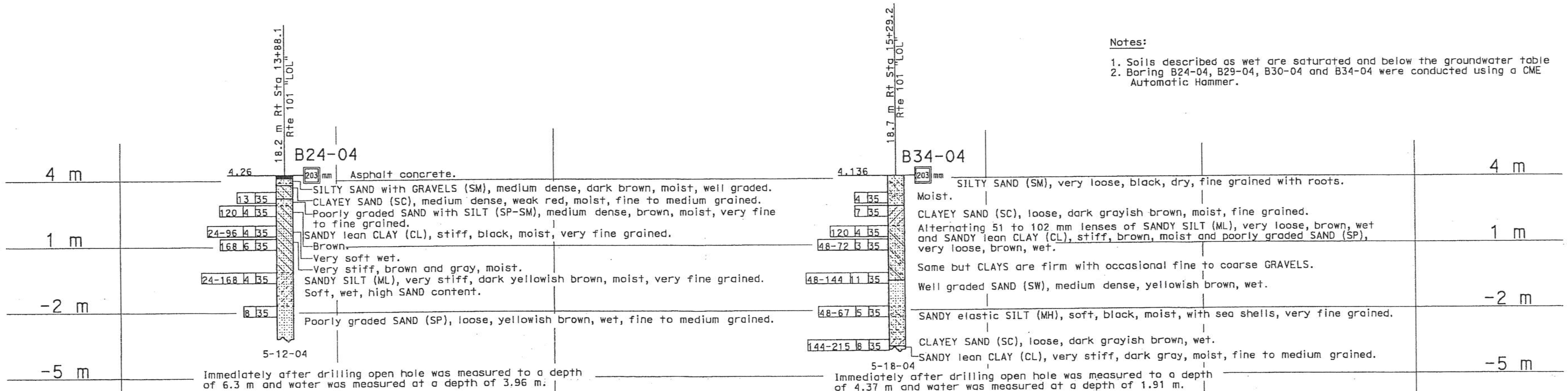
BENCH MARK

Sb 101 pm 12.09 Elev. 4.100 m
Sb 101 pm 12.19 Elev. 3.726 m

PLAN
1:1000

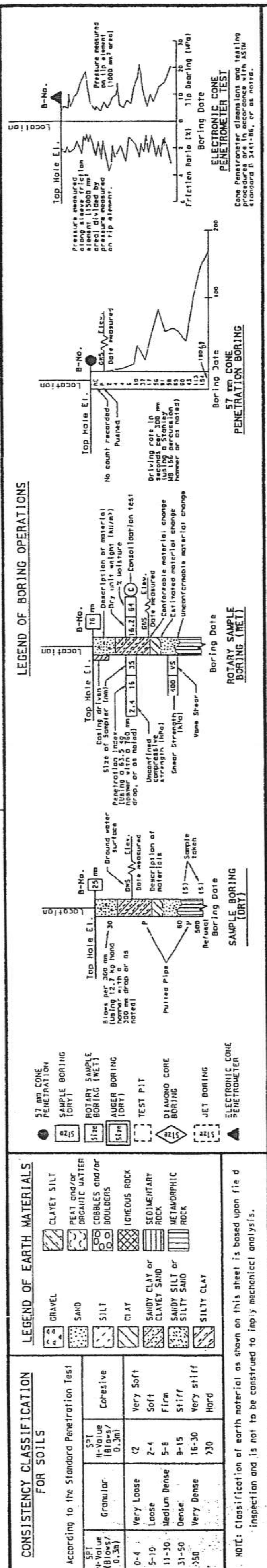
Notes:

1. Soils described as wet are saturated and below the groundwater table
2. Boring B24-04, B29-04, B30-04 and B34-04 were conducted using a CME Automatic Hammer.



PROFILE
HOR. 1:500
VER. 1:100

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN



ENGINEERING SERVICES		
DRAWN BY	W. Tang 09/04	
CHECKED BY	S. von Schwind	S. von Schwind

CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN

BRIDGE NO.
KILOMETER POST

SOUND WALL NO. 2
LOG OF TEST BORINGS 1 OF 2

CU 05
EA 447801

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 10-24-05 6-13-05 4-06-07

SHEET 9 OF 10

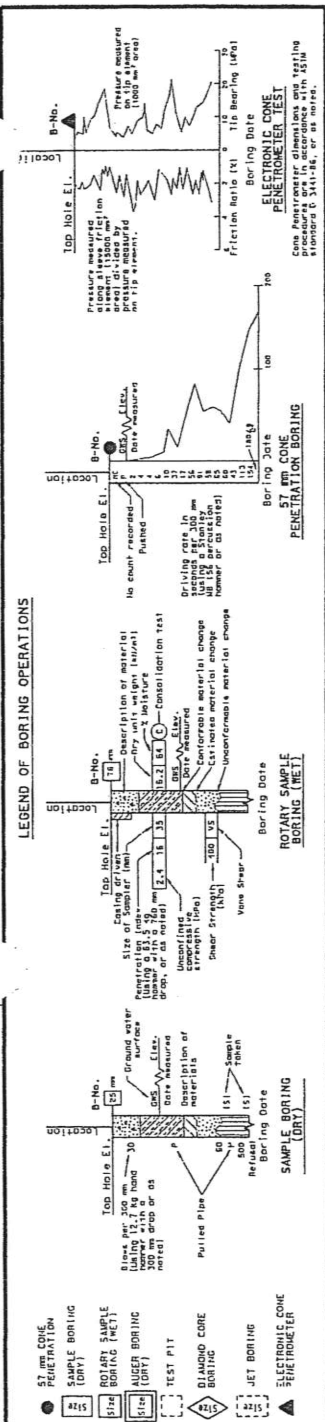
DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
05	SB	101	17.4/20.6	639	652

REGISTERED CIVIL ENGINEER 6-22-05

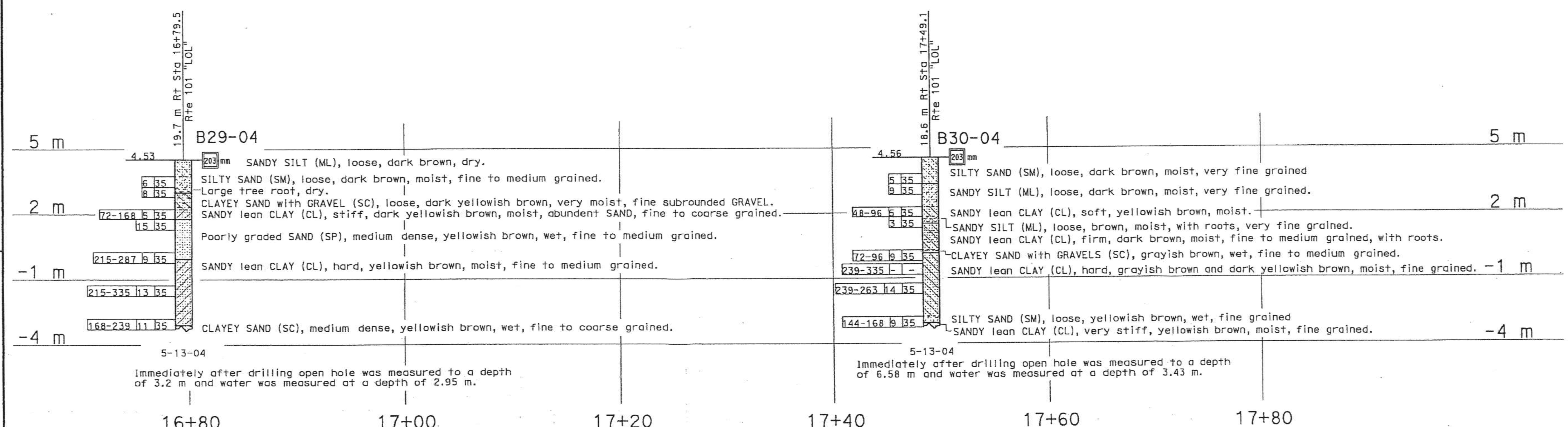
9-17-07
PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

REGISTERED PROFESSIONAL ENGINEER Sara von Schwind No. C050789 Exp. 9-30-05 CIVIL STATE OF CALIFORNIA



FOR PLAN VIEW, SEE
"LOG OF TEST BORINGS" 1 OF 2



PROFILE
HOR. 1:200
VER. 1:100

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

CONSISTENCY CLASSIFICATION FOR SOILS
According to the Standard Penetration Test

SPT N-Value (Blows/0.3m)	Granular	SPT N-Value (Blows/0.3m)	Cohesive
0-4	Very Loose	<2	Very Soft
5-10	Loose	2-4	Soft
11-30	Medium Dense	5-8	Firm
31-50	Dense	9-15	Stiff
>50	Very Dense	16-30	Very Stiff
		>30	Hard

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.

ENGINEERING SERVICES

DRAWN BY W. Tang 09/04
CHECKED BY S. von Schwind

S. von Schwind

CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN

BRIDGE NO.
KILOMETER POST

SOUND WALL NO. 2

LOG OF TEST BORINGS 2 OF 2

CU 05
EA 447801

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 05-04-05 5-13-05 4-06-07

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
05	SB	101	17.4/20.6	510	652

REGISTERED CIVIL ENGINEER 3-17-05

9-17-07
PLANS APPROVAL DATE

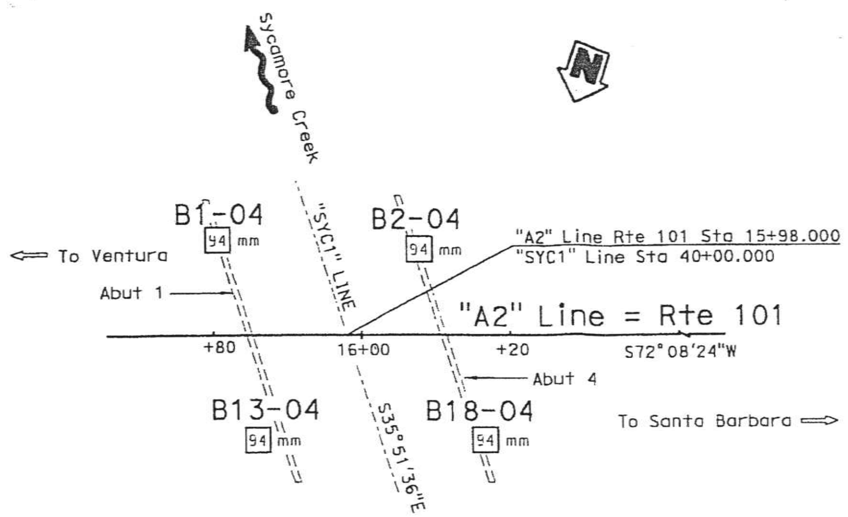
The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

REGISTERED PROFESSIONAL ENGINEER Sara von Schwind No. C050789 Exp. 9-30-05 CIVIL STATE OF CALIFORNIA

Caltrans Metric

BENCH MARK

SB101 pm12.30 = NAVD88 4.962 m
SB101 pm12.19 = NAVD88 3.726 m



PLAN

1:500

Note: Boring B1-04 was conducted using a CME Automatic Hammer.

B1-04

12.60 m Lt Sta 15+80.7
Rte 101 LOL

5.09 — 94 mm

- Asphalt concrete and base.
- CLAYEY SAND with GRAVELS (SC), loose, yellowish brown, moist, fine to medium grained SAND, fine subrounded GRAVELS.
- GWS El. 3.56 3-17-05 — No GRAVELS, dark brown.
- Very loose, brown, wet.
- Well graded SAND with GRAVEL (SW).
- CLAYEY SAND (SC), medium dense, dark brown, wet, fine grained.
- SANDY SILT (ML), medium dense, gray, wet, fine grained.
- CLAYEY SAND (SC), medium dense, gray, wet, fine grained.
- Gray and brown.
- Reddish brown (with thin lenses less than 25 mm of clean SAND)
- SILTY SAND (SM), medium dense, reddish brown, wet, fine grained.
- Lean CLAY (CL), hard, reddish brown, moist.
- CLAYEY SAND (SC), medium dense, reddish brown, moist.
- Poorly graded SAND (SP), medium dense, reddish brown, wet.
- Lean CLAY (CL), stiff, reddish brown, moist.
- CLAYEY SAND (SC), medium dense, reddish brown, wet.
- Well graded SAND (SW), medium dense, reddish brown, wet.
- SANDY SILT (ML), medium dense, reddish brown, wet.
- Poorly graded SAND (SP), medium dense, reddish brown, wet, fine grained.
- SANDY SILT (ML), medium dense, reddish brown, wet.
- Poorly graded SAND (SP), medium dense, reddish brown, wet, fine grained.
- SILTY SAND (SM), dense, reddish brown, wet, slightly cemented.
- Poorly graded SAND (SP), dense, reddish brown, wet, fine to medium grained.
- Fine grained.
- SILTY SAND (SM), dense, reddish brown, wet, slightly cemented.
- CLAYEY SAND with GRAVELS (SC), medium dense, wet, fine subrounded GRAVELS.
- Well graded SAND (SW), medium dense, reddish brown.
- SILTY SAND (SM), medium dense, reddish brown, wet, fine grained.
- Lean CLAY (CL), hard, reddish brown and gray, moist, highly plastic.
- SANDY lean CLAY.
- Lean CLAY.
- 76 mm well graded SAND lense, light yellowish brown, light gray and reddish brown.
- Poorly graded SAND (SP), dense, light brown, wet, very fine grained.
- CLAYEY SAND (SC), medium dense, light brown, wet, very fine grained.
- Poorly graded SAND (SP), medium dense, reddish brown, wet, fine grained.
- Very dense, yellowish brown.
- Medium to fine grained.

Blow counts / qu (kPa): 8 35; 3 35; 11 35; 15 35; 7 35; 16 35; 215; 144 17 35; 18 35; 40 35; 24 35; 287-431; 17 35; 407-431; 383-407 25 35; 359; 31 35; 16 35; 71 35; 57 35

5-4-04

B13-04

13.90 m Rt Sta 15+85.9
Rte 101 LOL

5.15 — 94 mm

- Asphalt concrete.
- SILTY SAND (SM), medium dense, dark brown, moist, fine SAND with occasional GRAVEL to 19 mm.
- SAND - well graded with GRAVEL.
- Elastic SILT with SAND (MH), loose, light brown, moist, very fine SAND, low to medium plasticity.
- Alternating SILT with SAND and well graded SAND with GRAVEL.
- SANDY elastic SILT (MH), medium dense, light brown, moist, low to medium plasticity.
- SILTY SAND (SM), light brown, moist, fine SAND.
- SANDY fat CLAY (CH), very stiff, tan/gray mottled, moist. qu= 335 to 407 kPa
- SILTY SAND (SM), well graded.
- Loose, light brown, moist, fine SAND.
- Medium dense, SAND fine to medium.
- Fat CLAY (CH), greyish brown, moist.
- SILTY SAND (SM), fine to medium SAND.
- Fat CLAY (CH), tan, moist.
- SILTY SAND (SM), medium dense, tan, moist.
- Fat CLAY (CH), tan, moist with SILTY SAND (SM) lenses.
- SILTY SAND (SM) to poorly graded SAND with SILT (SP-SM); dense, dark tan, moist.
- Fat CLAY (CH), hard, grey and brown mottled, moist, with GRAVEL occasionally.
- With very dense SILT lenses to 51 mm thick.
- SILTY SAND (SM), dense, tan, moist, fine to very fine SAND with occasional GRAVEL TO 19 mm.
- Fat CLAY (CH), hard, gray, moist, very plastic.
- CLAYEY SAND (SC), dense, dark tan, moist.
- SANDY fat CLAY (CH), hard, greyish brown, moist.
- No SAND.
- SANDY fat CLAY (CH), hard, grey and tan mottled, moist, medium plasticity, very fine grained.
- Fat CLAY (CH), hard, light gray with tan mottled, moist, medium plasticity.
- Elastic SILT with SAND (MH), very dense, gray with tan to red tan mottled, moist, very fine SAND.
- Poorly graded SAND with SILT (SP-SM), very dense, reddish tan, wet, fine SAND.
- Elastic SILT with SAND (MH), moist.
- Poorly graded SAND with SILT (SP-SM), very dense, dark tan, wet, fine SAND.
- With GRAVEL.
- Alternating SILTY SAND (SM) and SAND with SILT (SP-SM), dark tan, moist, with occasional GRAVEL to 25 mm.
- SILTY SAND (SM), very dense, dark tan, moist.

Blow counts / qu (kPa): 18 35; 6 35; 22 35; 335-407 17 35; 10 35; 18 35; 16 35; 45 35; >431; 28 35; >431; 44 35; 383-431 35 35; 192-383; >383; >431 47 35; 168->431; >431 51 35; >62 35; 48/0.152 35; >50 35; >50 35

5-11-04

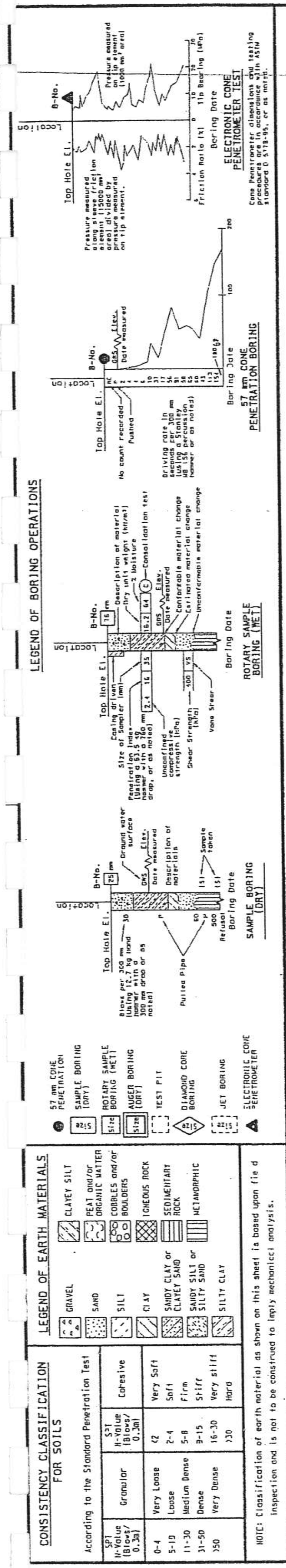
Elevations: 5 m, 2 m, -1 m, -4 m, -7 m, -10 m, -13 m, -16 m, -19 m, -22 m

Stations: 15+80, 15+84, 15+88

PROFILE

HOR. 1:20
VER. 1:100

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN



NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.

ENGINEERING SERVICES	GEOTECHNICAL SERVICES	FIELD INVESTIGATION BY:
DRAWN BY: F. Nguyen 9/04		
CHECKED BY: S. von Schwind		S. von Schwind, W. Hoon

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN 6

BRIDGE NO. 51-0332
KILOMETER POST 19.8

SYCAMORE CREEK BR (REPLACE)

LOG OF TEST BORINGS 1 OF 2

CU 05
EA 447801

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

SHEET 18 OF 19

PLOTTED => 21-SEP-2007 TIME PLOTTED => 09:02 USERNAME => friend

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
05	SB	101	17.4/20.6	511	652

REGISTERED CIVIL ENGINEER 3-17-05

9-17-07
PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

REGISTERED PROFESSIONAL ENGINEER — Sara von Schwind — No. C050789 — Exp. 9-30-05 — CIVIL — STATE OF CALIFORNIA

Caltrans Metric

FOR PLAN VIEW, SEE "LOG OF TEST BORINGS" 1 OF 2

B2-04

11.30 m Lt Sta 16+07.9 Rte 101 LOL

Elev. 5.26 — 94 mm Asphalt concrete.

- SANDY SILT (ML), loose, brown, moist, fine SAND, non-plastic. (6 | 35)
- SANDY CLAY (CL), stiff, brown, moist, fine SAND, medium plasticity. (96-144 | - | -) (8 | 35)
- SILTY SAND (SM), medium dense, brown, moist, fine, uniformly graded, non-plastic. (12 | 35) (13 | 35)
- CLAYEY SAND (SC), medium dense, brown, moist, fine SAND, low plasticity.
- SILTY SAND (SM), medium dense, brown, moist, fine, trace fine GRAVEL, uniformly graded, non-plastic. (20 | 35)
- Lean CLAY (CL), stiff, brown, moist, medium plasticity. (96 | 10 | 35)
- Poorly graded SAND (SP), medium dense, brown, moist, fine to medium graded, non-plastic. (18 | 35)
- CLAYEY SAND (SC), medium dense, brown, moist, fine SAND, low plasticity.
- SILT with SAND (ML), medium dense, brown, moist, fine, uniformly graded, non-plastic. (24 | 35)
- Poorly graded SAND (SP), dense, brown, moist, fine, trace of fine subrounded GRAVEL, non-plastic, lense of GRAVEL at elev -8.15 m. (35 | 35)
- Lense of lean CLAY from elev -8.70 to elev -8.76 m, stiff, brown, moist, low plasticity. (15 | 35)
- Lean CLAY (CL), hard, brown, moist, medium plasticity. (215 | - | -)
- SILTY SAND (SM), medium dense to dense, brown, moist, fine SAND, non-plastic. (28 | 35)
- Hard drilling from elev -13.88 m to -14.25 m. (34 | 35)
- Lean CLAY (CL), hard, greenish gray, moist, medium plasticity. (431 | 39 | 35)
- Poorly graded SAND with GRAVEL (SP), dense, light brown, moist, fine SAND, fine rounded GRAVEL, low plasticity.
- SILTY SAND (SM), very dense, tan, moist, fine SAND, non-plastic. (82 | 35)
- Poorly graded SAND (SP), very dense, light brown, moist, fine, uniformly graded, non-plastic. (48 | 35)
- CLAYEY SAND (SC), dense, light green, moist, fine, low plasticity.
- Poorly graded SAND (SP), very dense, light brown, moist, fine to medium grained, non-plastic. (>70 | 35) (>70 | 35)

5-03-04

B18-04

13.90 m Rt Sta 16+16.6 Rte 101 LOL

Elev. 5.26 — 94 mm Asphalt concrete.

- SILT (ML), loose, brown, moist, some very fine SAND and ORGANICS throughout. (5 | 35)
- SILTY SAND (SM), medium dense, dark tan, moist, poorly graded fine SAND. (33 | 35)
- With GRAVEL, gray, GRAVEL in tip - well rounded 25 mm, slight organic odor and some small roots (black).
- SANDY elastic SILT (MH).
- Fat CLAY (CH), very stiff, dark tan, moist, medium plasticity. (215-287 | 17 | 35)
- CLAYEY SAND (SC), medium dense, dark reddish tan, moist, SAND is well graded. (20 | 35)
- SANDY SILT (MH), medium dense, dark tan, moist. (20 | 35)
- Fat CLAY (CH), dark tannish gray.
- SANDY SILT (MH), medium dense, dark tan, moist (25 | 35)
- SILTY SAND (SM), medium dense, tan, moist.
- SANDY elastic SILT (MH).
- SILTY SAND (SM), medium dense, tan, moist, SAND is fine to medium. (16 | 35)
- SANDY elastic SILT (MH), dark reddish tan, moist, fine grained.
- Poorly graded SAND (SP), dense, dark tan, moist. (40 | 35)
- SILTY SAND (SM), dense, dark tan, moist, SAND is fine to very fine grained.
- With GRAVEL to 25 mm, well rounded.
- Varying amount of SILT/SAND.
- Lean CLAY (CL), soft, tan, moist, medium plasticity. (<48 | 14 | 35)
- CLAYEY SAND (SC), medium dense, dark tan, moist, fine to medium grained.
- SILTY SAND (SM), dark reddish tan, moist.
- SANDY fat CLAY (CH), brown, moist.
- Lean CLAY (CL), hard, brown, moist. (359->431 | 33 | 35)
- CLAYEY SAND (SC), medium dense, tan to gray, moist, with occasional 13 mm GRAVEL, fine grained. (29 | 35)
- SANDY lean CLAY (CL), hard, gray/tan mottled. SAND is very fine and ribboned throughout sample, medium plasticity. (215->431 | - | -)
- SILTY SAND (SM), dark tan, moist. (359 | 22 | 35)
- Fat CLAY (CH), hard, gray, moist, high plasticity.
- SANDY lean CLAY (CL), dark reddish tan with red brown mottled, moist, very fine grained. (215-407 | - | -)
- Fat CLAY (CH), hard, gray, moist. (25 | 35)
- SILTY SAND (SM), dense, gray and tan mottled, moist, fine to very fine grained. (37 | 35)
- Very dense. (>50 | 35)
- Weak cementation. (>50 | 35)
- With some GRAVEL to 51 mm.
- With very occasional GRAVEL to 13 mm. (>50 | 35)

5-12-04

Elevations: 5 m, 2 m, -1 m, -4 m, -7 m, -10 m, -13 m, -16 m, -19 m, -22 m, -25 m

Stations: 15+80, 16+00, 16+20, 16+40

PROFILE
HOR. 1:100
VER. 1:100

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

LEGEND OF BORING OPERATIONS

LEGEND OF EARTH MATERIALS

CONSISTENCY CLASSIFICATION FOR SOILS

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.

ENGINEERING SERVICES	GEOTECHNICAL SERVICES	FIELD INVESTIGATION BY:	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF STRUCTURES STRUCTURE DESIGN 6	BRIDGE NO. 51-0332	SYCAMORE CREEK BR (REPLACE)
DRAWN BY F. Nguyen 9/04		S. von Schwind			KILOMETER POST 19.8	LOG OF TEST BORINGS 2 OF 2
CHECKED BY S. von Schwind		W. Hoon, D. Appelbaum		CU 05 EA 447801		SHEET 19 OF 19

APPENDIX D
LOG OF TEST BORING SHEET

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
5	SB	Punta Gorda St.	————————	1	3

REGISTERED GEOTECHNICAL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GREGORY S. DENLINGER No. 2249 Exp. 3/31/11 Geotechnical STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

Fugro West, Inc.
4820 McGrath Street, Suite 100
Ventura, CA 93003

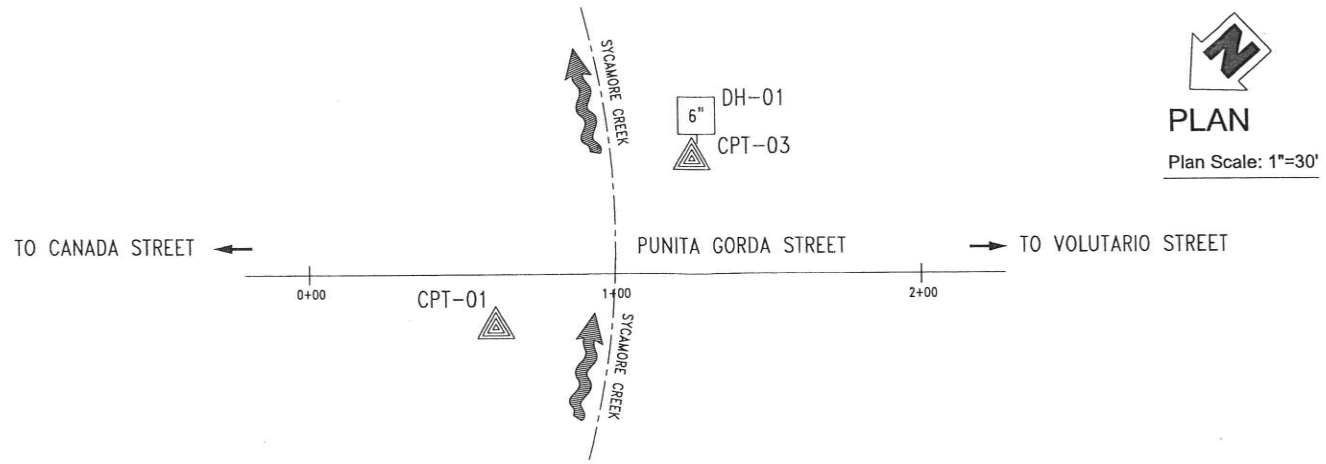
Penfield & Smith Engineers
111 East Victoria Street
Santa Barbara, CA 93101

Benchmark:

Lead and Tag, "CITY ENGR", Northwesterly Corner, Punita Gorda Street and Canada Street

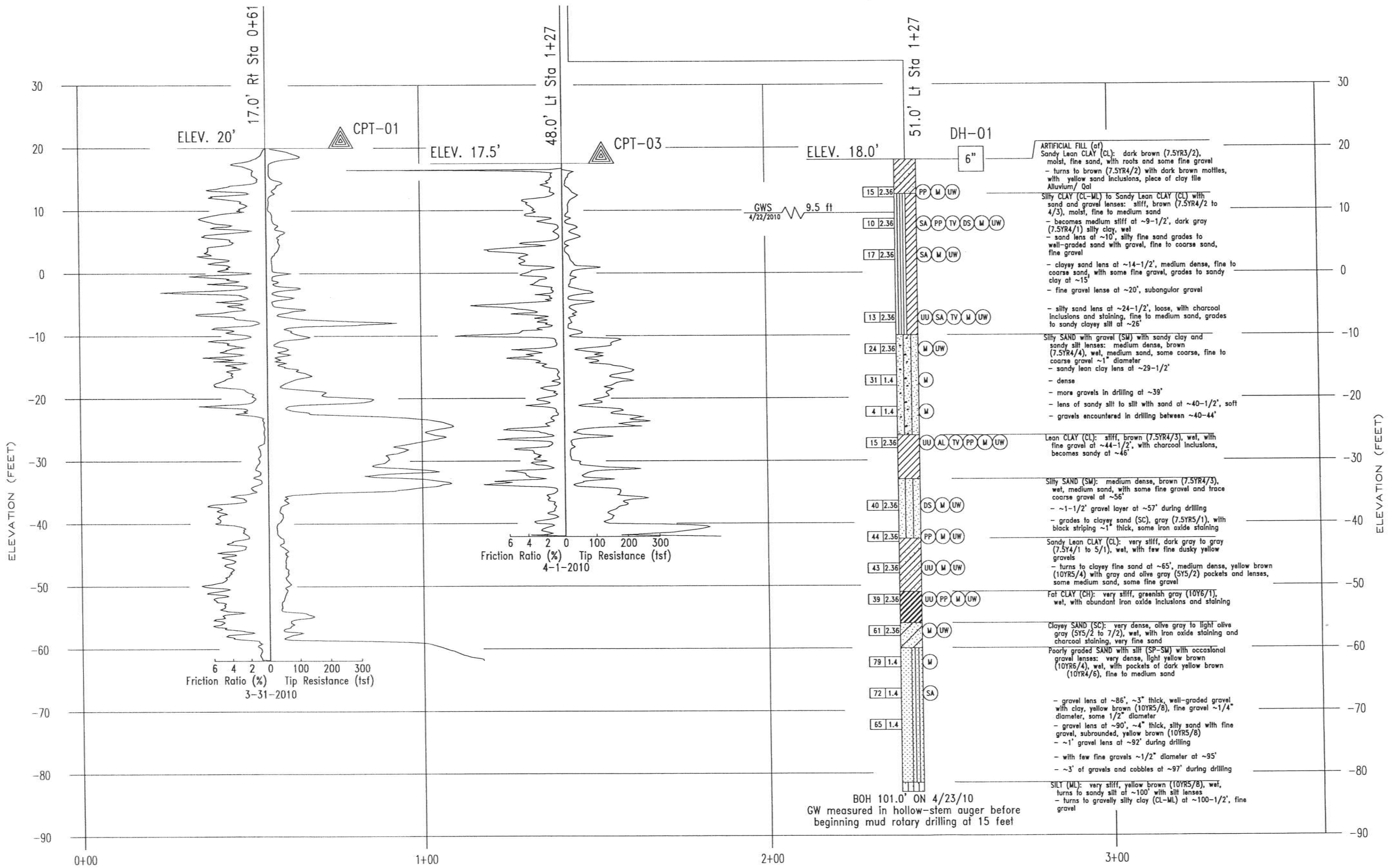
Elevation: 16.78' NAVD 88

Stationing: assumed on Punta Gorda Street with Station 1+00 at approx. center of bridge



PROFILE

Profile Scale: 1"=10' vertical
1"=50' horizontal



NOTES:

1. Mud rotary borings excavated with a CME-75 drill rig equipped with an automatic trip hammer weighing 140 pounds falling 30 inches.
2. Standard Penetration Test Samples I.D.=1-3/8 in., O.D.=2 in., without liners.
3. "2.4" sampler has a 2-3/8 in. inside diameter and a 3 in. outside diameter (Modified Calif. Sampler).
4. CPT Sounding performed with 20-ton truck rig.
5. "ref/2" " Drive exceeded 50 blows during initial 6 in. seating
6. "90/11" Partial drive having number of blows over depth interval noted.
7. This LOTB was prepared in accordance with the Caltrans Soil & Rock Logging, Classification and Presentation Manual (2010).

M:\Drafting\JOBFILES\2010\3037\3037.047\Drawings\3037.047 LOTB main.dwg 07-20-2010 - 5:30pm

TIME PLOTTED -) $TIME
DATE PLOTTED -) $DATE
USERNAME -) $USER

ENGINEERING SERVICES		GEOTECHNICAL SERVICES	City of Santa Barbara Public Works	Penfield & Smith Engineers	BRIDGE NO. N/A	PUNTA GORDA STREET BRIDGE (REPLACE)
FUNCTIONAL SUPERVISOR	DRAWN BY: B. Egan	FIELD INVESTIGATION BY:			POST MILES N/A	LOG OF TEST BORINGS
NAME: C. Prentice	CHECKED BY: G. Denlinger	K. Robinson				

OGS CIVIL LOG OF TEST BORINGS SHEET

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU ——
EA ——

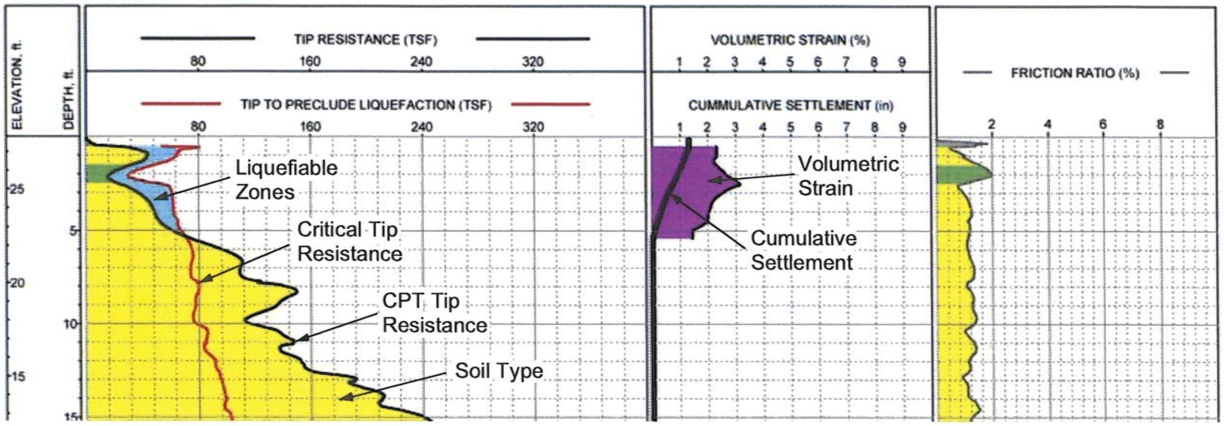
FILE => $REQUEST

DISREGARD PRINTS BEARING EARLIER REVISION DATES

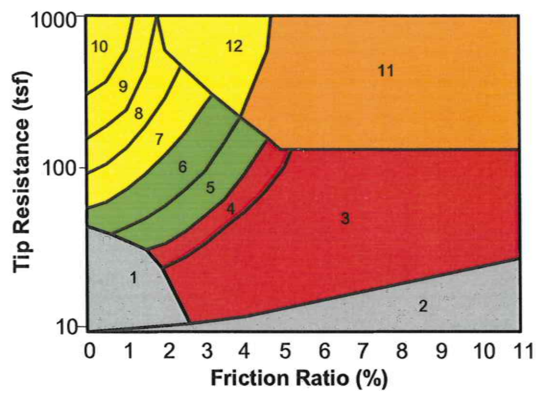
REVISION DATES

SHEET 1 OF 3

APPENDIX E
LIQUEFACTION EVALUATION



COLOR LEGEND FOR FRICTION RATIO TRACES



Zone	Soil Behavior Type	U.S.C.S.
1	Sensitive Fine-grained	OL-CH
2	Organic Material	OL-OH
3	Clay	CH
4	Silty Clay to Clay	CL-CH
5	Clayey Silt to Silty Clay	MH-CL
6	Sandy Silt to Clayey Silt	ML-MH
7	Silty Sand to Sandy Silt	SM-ML
8	Sand to Silty Sand	SM-SP
9	Sand	SW-SP
10	Gravelly Sand to Sand	SW-GW
11	Very Stiff Fine-grained *	CH-CL
12	Sand to Clayey Sand *	SC-SM

*overconsolidated or cemented

CPT CORRELATION CHART

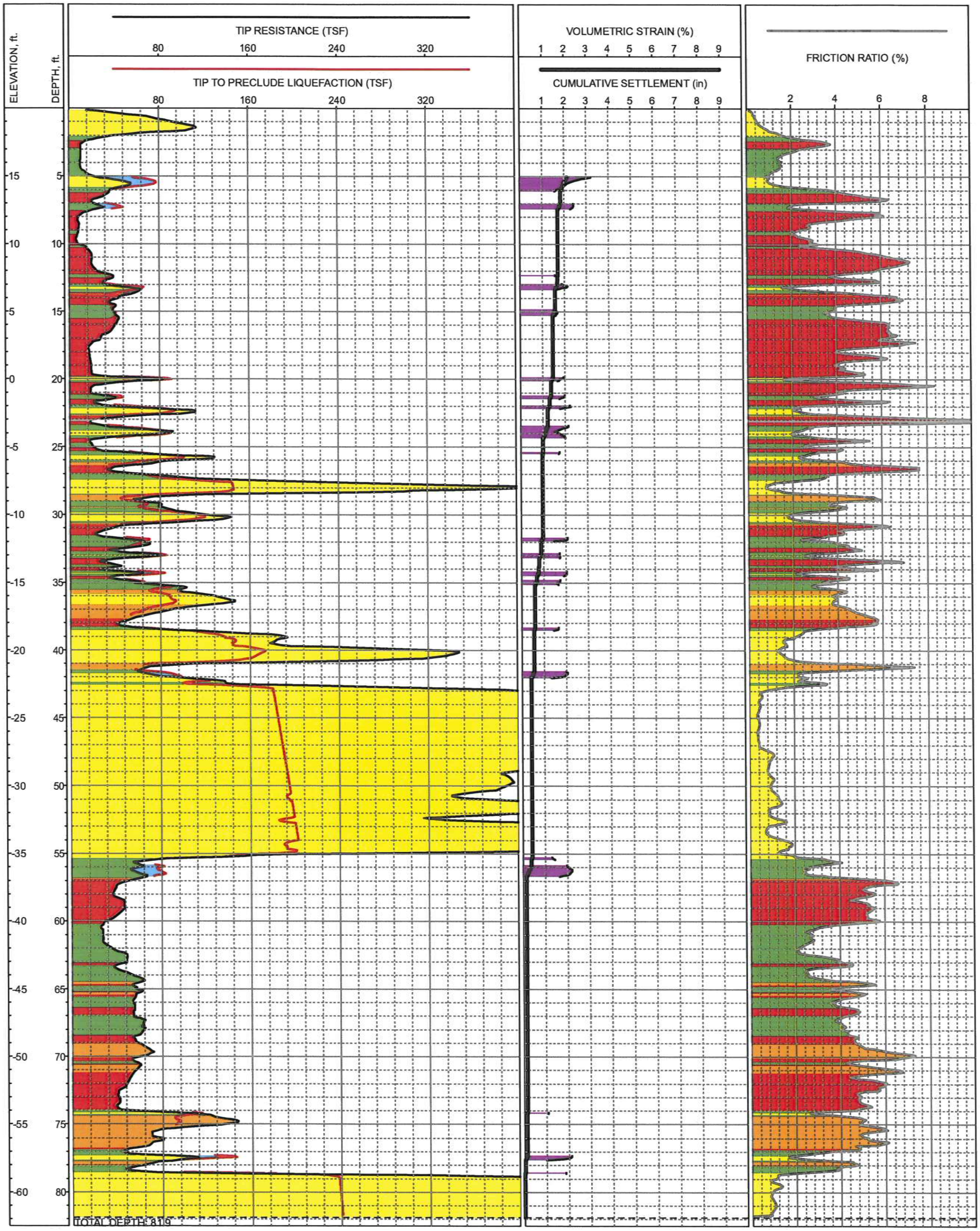
(Robertson and Campanella, 1988)

KEY TO LIQUEFACTION LOGS

Sycamore Creek Enhancement Project
Santa Barbara, California

N:\Projects\3037_PenfieldSmith\3037-047_SycamoreCreek\Explorations\CPT2010\Logs\2010_05_10_Logs_M7_2_a0_64\MXD\Plate_E5_KeytoCPT-Liquefaction.mxd, 07/27/10, CDean

Penfield & Smith Engineers
Project No. 3037.047



LOCATION:
SURFACE EL: 20.0ft +/- (MSL)
COMPLETION DEPTH: 81.9ft
TESTDATE: 3/31/2010

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: K Robinson

LOG OF CPT NO: CPT-1

M = 7.2, P.G.A. = 0.64g

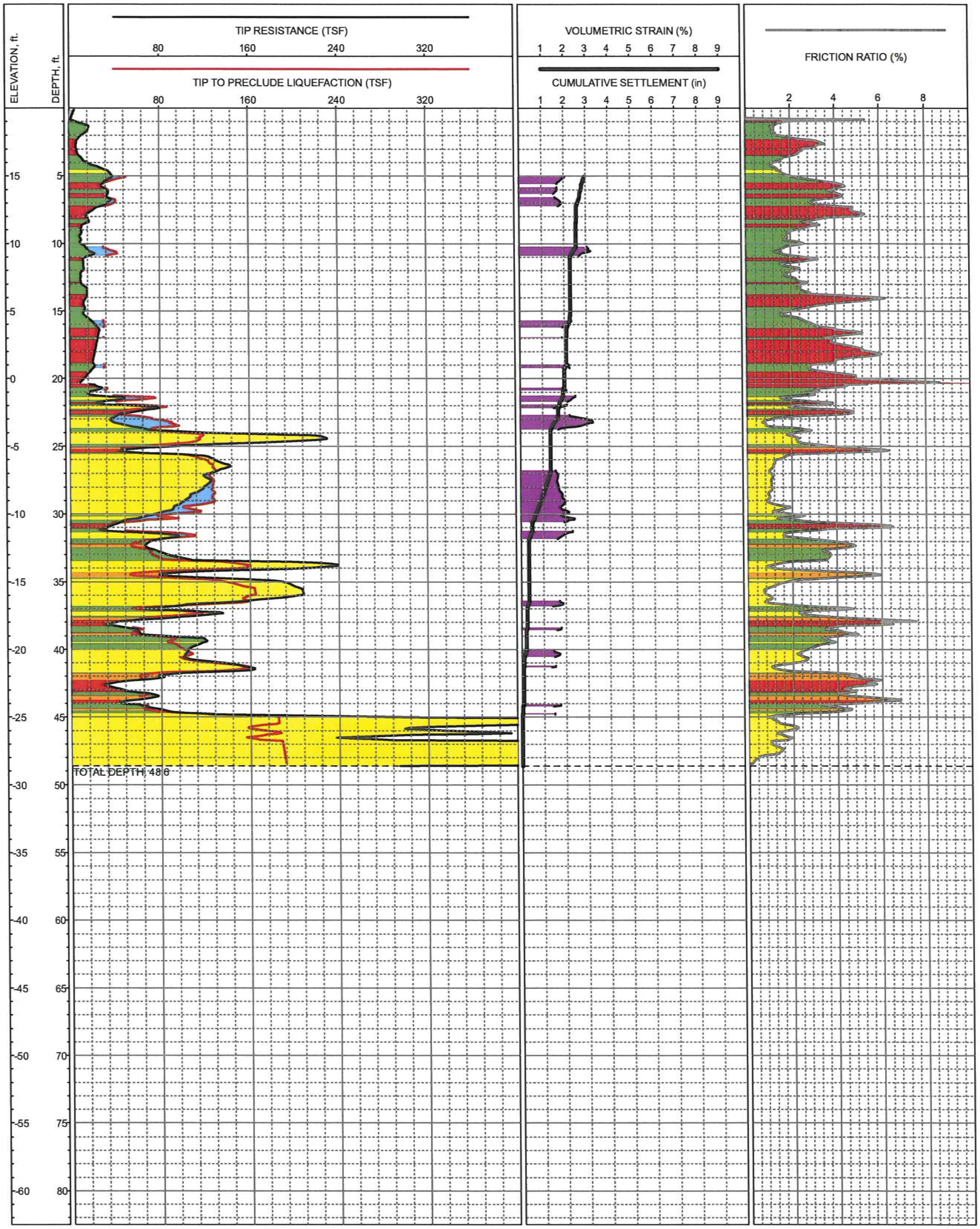
Sycamore Creek Enhancement Project

Santa Barbara, California

PLATE A1

N:\Projects\3037_PenfieldSmith\3037-047_SycamoreCreek\Explorations\CPT2010\Logs\2010_05_10_Logs_M7_2_a0_64\MXD\Logs_M7_2_a0_64.mxd,05/10/2010,CDean

Penfield & Smith Engineers
Project No. 3037.047



LOCATION:
SURFACE EL: 20.0ft +/- (MSL)
COMPLETION DEPTH: 48.6ft
TESTDATE: 3/31/2010

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: K Robinson

LOG OF CPT NO: CPT-2

M = 7.2, P.G.A. = 0.64g

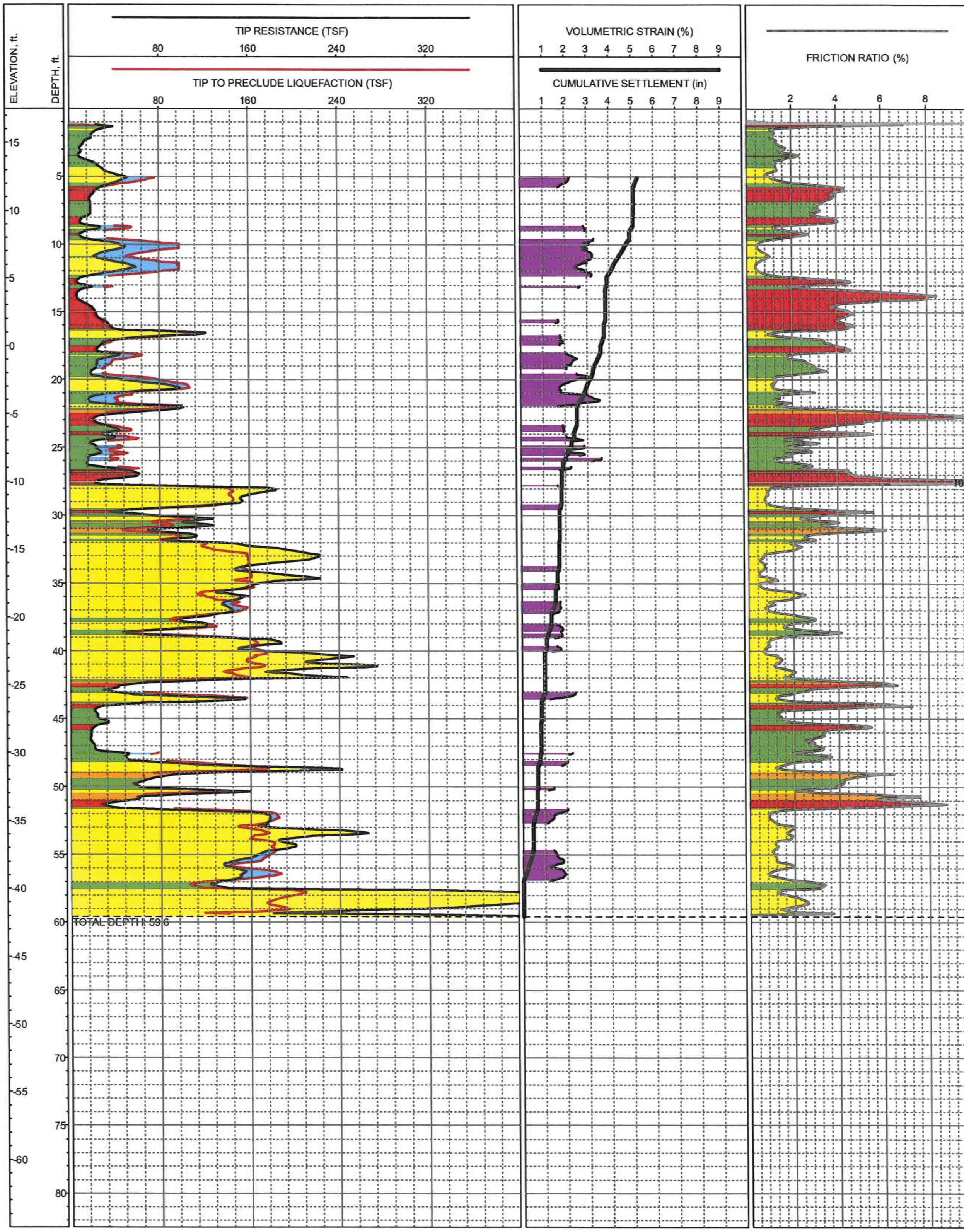
Sycamore Creek Enhancement Project

Santa Barbara, California

PLATE A2

N:\Projects\3037_PenfieldSmith\3037-047_SycamoreCreek\Explorations\CPT\2010\Logs\2010_05_10_Logs_M7_2_a0_64\MXD\Logs_M7_2_a0_64.mxd,05/10/2010,CDean

Penfield & Smith Engineers
Project No. 3037.047



LOCATION:
SURFACE EL: 17.5ft +/- (MSL)
COMPLETION DEPTH: 59.6ft
TESTDATE: 4/1/2010

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: K Robinson

LOG OF CPT NO: CPT-3

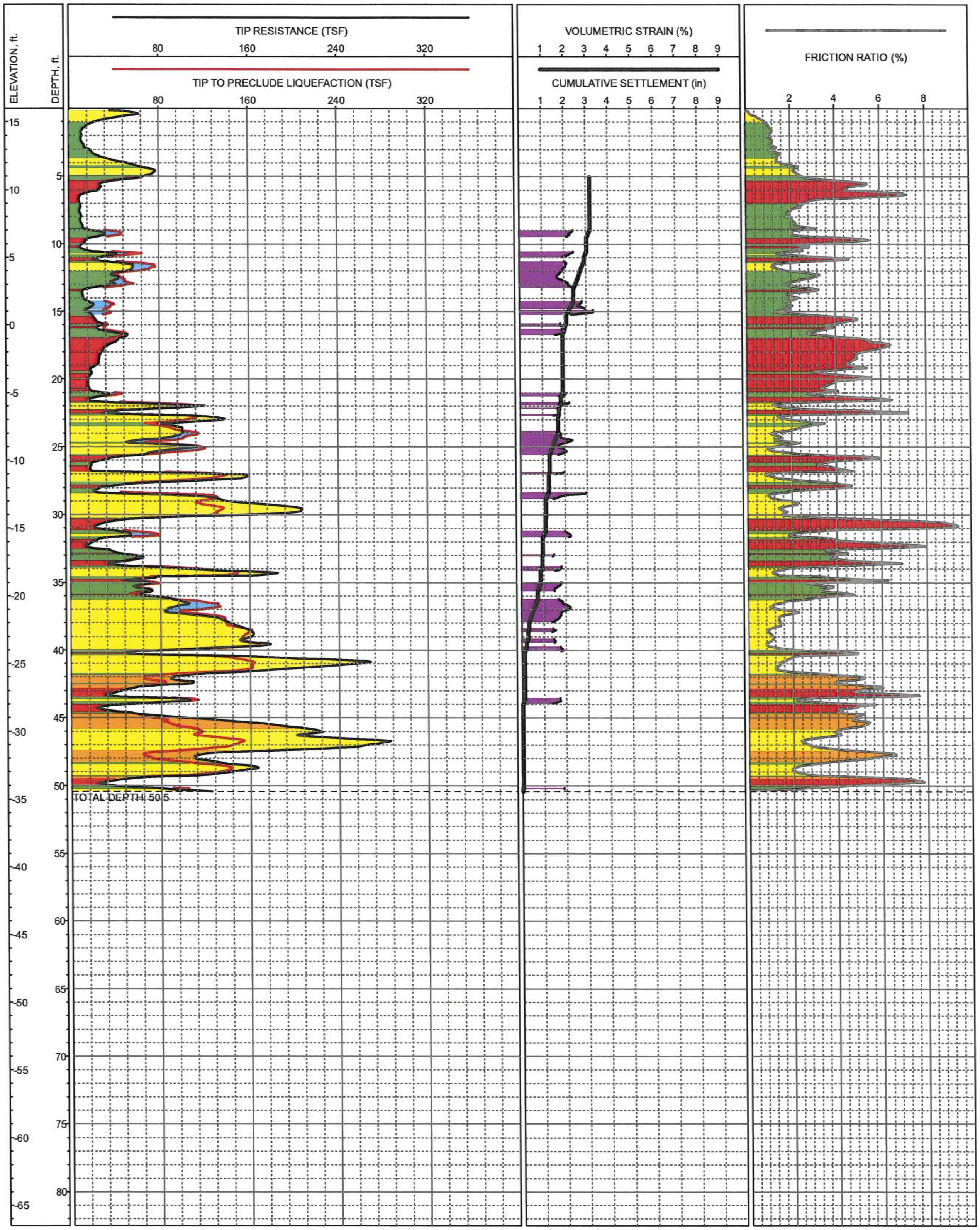
M = 7.2, P.G.A. = 0.64g

Sycamore Creek Enhancement Project

Santa Barbara, California

PLATE A3

N:\Projects\3037_PenfieldSmith\3037-047_SycamoreCreek\Explorations\CPT\2010\Logs\2010_05_10_Logs_M7_2_a0_64\MXD\Logs_M7_2_a0_64.mxd,05/10/2010,CDean



LOCATION:
SURFACE EL: 16.0ft +/- (MSL)
COMPLETION DEPTH: 50.5ft
TESTDATE: 4/1/2010

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: K Robinson

LOG OF CPT NO: CPT-4

M = 7.2, P.G.A. = 0.64g

Sycamore Creek Enhancement Project

Santa Barbara, California

PLATE A4

N:\Projects\3037_PenfieldSmith\3037-047_SycamoreCreek\Explorations\CPT2010\Logs\2010_05_10_Logs_M7_2_a0_64\MXD\Logs_M7_2_a0_64.mxd,05/10/2010,CDean